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Deep Tunnels in Hard Rock

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SELECTED WATER RESOURCES ABSTRACTS
INPUT TRANSACTION FORM

1. Report No.

2.

3. Accession No. **W** 72-10834

4. *Title* DEEP TUNNELS IN HARD ROCK,

5. Report Date

6.

7. *Author(s)*

8. Performing Organization Report No.

9. *Organization* Wisconsin University, Milwaukee, College of Applied Science and Engineering

10. Project No. EPA Program 110[illegible] (02/71)

11. Contract/Grant No.

12. ~~Sponsoring Organization~~ Water Pollution Control Research Series ~~[illegible]~~

13. Type of Report and Period Covered

15. *Supplementary Notes* Proceedings of an engineering institute, Wisconsin University, Milwaukee, November 9-10, 1970. Environmental Protection Agency~~, Office of Research and Monitoring, Washington, D.C.~~, February 1971. 221 p , 92 fig, 5 tab, 17 ref.

16. *Abstract* An institute was held to discuss tunneling in hard rock, deep below ground surfaces. The goals were to: describe tunneling experiences in Chicago, western United States, Canada, and Europe; discuss some of the multiple-use aspects of deep tunnels; assess the future of rapid excavation in hard rock using mining machines; and encourage those facing combined sewer and flooding problems to assess the potential use of tunnels. The flooding and combined-sewer overflow problem is discussed, as are the investigations and plans of the Metropolitan Sanitary District of Greater Chicago for constructing a deep tunnel system to alleviate flooding and pollution from combined-sewer overflows. The potential of pumped-storage hydroelectric power generation and circulation of condenser water from underground nuclear generating plants, in conjunction with tunnel construction, is treated. Experiences with mechanical moles in the United States and other countries in boring through hard rock are described. Suggestions are given concerning pre-construction investigations, planning and design that can minimize field problems, expedite tunneling procedures, and reduce tunneling costs. Various viewpoints are presented including those of the contractor, machine manufacturer, tunnel designer, utility company, geologist, and public agency. Eleven papers are included in this ~~proceedings~~ publication, compiled under the supervision of Professors Vinton W. Bacon and Paul A. Seaburg. (Poertner)

(See W72-10835 thru W72-10845)

17a. *Descriptors* *Tunneling, *Rock excavation, *Combined sewers, *Tunneling machines, *Underground storage, *Overflow, Tunnel construction, Storage, Sewerage, Flood control, Water pollution control, Urban runoff, Pumped storage, Nuclear powerplants, Urbanization, Underground powerplants, Storm drains, Multiple-purpose projects, Water management (applied), Aquifer management, Waste water treatment, Sanitary engineering, Rock mechanics,

17b. *Identifiers* *Deep tunnels, *Combined sewer overflows, Chicago, Metropolitan Sanitary District of Greater Chicago

Copy available from GPO Sup Doc EP2.10:11030 -- 02/71, $1.75 ; microfiche from NTIS as PB 210 854, $0.95.

17c. *COWRR Field & Group* 08H, 08A, 08C, 08E, 05D, 05G, 04A, 03C

18. Availability

19. Security Class. (Report)

20. Security Class. (Page)

21. No. of Pages

22. Price

Send To:

WATER RESOURCES SCIENTIFIC INFORMATION CENTER
U.S. DEPARTMENT OF THE INTERIOR
WASHINGTON, D. C. 20240

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Proceedings
from

DEEP TUNNELS IN HARD ROCK
A Solution to Combined Sewer Overflow and Flooding Problems

An Engineering Institute
Presented By

College of Applied Science and Engineering
The University of Wisconsin-Milwaukee

and

University Extension
The University of Wisconsin

Civic Center Campus
November 9-10, 1970
Milwaukee, Wisconsin

For sale by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 - Price $1.75

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ABSTRACT

The Proceedings which follow contain the information presented at an institute held at the Civic Center Campus of the University of Wisconsin Milwaukee (UWM), on November 9-10, 1970. The program was conducted by the University Extension under the technical guidance of the College of Applied Science and Engineering at UWM. Arrangements for the program and compilation of these Proceedings were completed under the supervision of Professors Vinton W. Bacon and Paul A. Seaburg.

These proceedings are published by the Office of Research and Monitoring, Environmental Protection Agency.

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Session 1

PROBLEM DEFINITION AND CURRENT SOLUTIONS

Moderator - W. A. Rosenkranz

Section 1

THE FLOODING AND COMBINED SEWER OVERFLOW PROBLEM IN URBAN METRO AREAS

by

Vinton W. Bacon
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THE FLOODING AND COMBINED SEWER OVERFLOW PROBLEM IN URBAN METRO AREAS

What is the magnitude of the combined-sewer problem in the United States? The most authoritative estimate in the U. S. was made in 1967 by the American Public Works Association under the sponsorship of the Federal Water Quality Administration of which our moderator, Mr. William Rosenkranz, is representative.

It is estimated that in the U. S. there are 1,329 jurisdictions, served in whole or in part by combined sewers, having a total population of 54 million. Of this projected population, it is estimated that 36 million are actually served by combined sewers. Typical are the heavily built-up, central cores in cities such as Milwaukee, Chicago, Cleveland, Minneapolis-St. Paul, Washington, D.C., New York, Boston, San Francisco, St. Louis, and many, many others.

Although the gallonage of sewage overflow is only about 5% of the total, it is estimated that about 30% of the total pollution material is overflowed to the waterway. This occurs because large storm sewers are laid on flat grades, causing low velocity and settling of much of the sewage solids within the pipes. The high velocities of storm flows scour up the material, carrying it to the waterway with the overflow. Thus in magnitude, tremendous quantities of pollutants are flushed from combined sewers.

The combined sewer overflow problem can be solved in one of three ways, or a combination of the three.

First, sewers can be separated, that is, a second sewer can be constructed in the street. In built-up cities, this would take years to complete and construction would occur in all of the combined sewer areas. Politically, it is doubtful if administrations attempting this solution would survive more than one term. Further, it is extremely costly. APWA estimates that if all jurisdictions were to solve the problem through separation that the total expenditure would approximate $30 billion and to make the necessary changes in and on private property to effect total separation would increase this total cost to approximately $48 billion. Responses from many of the municipalities surveyed, especially those with high population densities, disclose that the possibility of changing all combined sewers to separate is remote.

Chicago alone estimates that the cost of separation together with the property connections would cost in excess $4 billion to eliminate

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the effects of the 400 overflow points. Chicago has concluded not to try separation in the combined area.

Secondly, treatment can be provided at the point of overflow simply by interception before discharge. Treatment could be by primary settling, screening, aeration, disinfection, and other means. A number of worthy demonstration projects for this system are underway throughout the U. S., including Milwaukee. Because the overflow points are usually in built-up areas, one of the difficulties is the availability of land. This is likely to limit this application.

Having seen sewer separation as politically impractical and too costly, and having seen limitations of overflow interception and treatment, both Boston and Chicago, studying independently, developed an innovative solution. Both cities looked to the underground for space for the solution, and both separately came to the same conclusion: build conveyance tunnels (sewers) and storage caverns in the underground rock. By building tunnels under the present rivers in Chicago the overflow at 400 points can be intercepted by vertical drop shafts, thus allowing only "clean" surface runoff to the river system. The polluted combined sewer overflow would be stored during the storm, pumped back to the surface after the storm, and treated in existing or new plants. Nothing really new, except the configuration of the component parts. Underground pumping stations, room-and-pillar mining, circular tunnels mined in hard rock with mechanical mining machines, and other features have all been used and proved elsewhere.

This Institute on "Deep Tunnels in Hard Rock - A Solution to Combined Sewer Overflow and Flooding Problems" is based on 5 major convictions:

1. The combined-sewer overflow problem, with the attendant flooding problem, in metropolitan urban areas must be solved if the new and stringent State and Federal water quality standards are to be met. Secondary, tertiary, and advanced waste-water treatment of dry-weather flows will be a waste of money if the overwhelming raw-waste pollution of combined-sewer overflows is not likewise controlled.
2. Of the three apparent solutions (sewer separation, retention and treatment at the overflow points, and underground conveyance and storage), the latter provides the cheapest and most complete solution where rock strata conditions are favorable.

   Retention and treatment at overflow points, such as being developed and demonstrated here in Milwaukee, either separately or in combination with tunnel and storage facilities, has great merit, too.

Sewer separation in older, highly developed, and congested urban areas appears to have little to recommend it.

3. Key to the underground conveyance and storage solution is rapid and less costly excavation of tunnels in hard rock.
4. Key to such rapid excavation is the mechanical mining machine for circular or other tunnel configurations.
5. And lastly, the mechanical mining machine has demonstrated, possibly in a fledgling manner, that it can provide more rapid excavation and at less cost then other methods heretofore standard in tunnel construction.

With such convictions, what are the hoped-for goals of this Institute?

First, we want to hear of the successful recent experiences in Chicago, Western United States, Canada, Europe, and elsewhere. Through the mere recitation of such experience sharing, we hope to give impetus to further development and application and cost reduction.

Secondly, we hope to develop some of the multiple-use aspects of the deep-tunnel solutions, such as, pumped storage for hydroelectric storage, storage as a substitute for waste treatment plant capacity, the impact of storage on the water resources of an area, and the impact of the system on recreation, navigation, and other beneficial uses.

Third, we hope to assess the future of rapid excavation in hard rock by mechanical mining through the opinions of machine manufacturers, construction contractors, directors of public work, consultants, educators, and government.

Lastly, we hope to encourage those facing combined sewer and flooding problems to assess the potential use of tunnels in their areas, believing that this system is in its infancy and needful of the creative thinking of many. Boston, as Chicago, has concluded, after extensive engineering studies that rock tunnels provide the solution for that metropolitan area.

Section 2

METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO EXPERIENCES AND FUTURE PLANS FOR HARD ROCK TUNNELS

by

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GENERAL INFORMATION

The Metropolitan Sanitary District of Greater Chicago is a municipal corporation serving 5,500,000 persons living within a 860-square mile area in Cook County, Illinois. There are approximately 118 municipalities, including the City of Chicago, and 30 sanitary districts under its jurisdiction.

Early sewer systems, developed prior to the turn of the century to serve the City of Chicago and the peripheral suburbs, were combined. They discharged untreated flows directly into the waterways, which, in turn, flowed into Lake Michigan - polluting the source of water supply for Chicago and many of the suburbs.

To protect the water supply, the Metropolitan Sanitary District was organized in 1889. Drainage in the Chicago River Basin and the Calumet River Basin was diverted from Lake Michigan (St. Lawrence River Watershed) into the Des Plaines River (Mississippi Watershed). This was accomplished by construction of the Sanitary and Ship Canal in 1911, which served the northern suburbs of Chicago; and the Cal Sag Channel in 1923, which reversed the Calumet River System. Three control structures prevent the river and canal system from flowing into the lake and permit the entrance of lake water to the system. The canal system, in addition to providing pollution and flood control, is a major shipping artery for bulk commodities. (Fig. 1)

At one time up to 9,000 cfs were diverted from Lake Michigan into the canal system to dilute the sewage. In 1925 several states instituted a suit against the State of Illinois and the Metropolitan Sanitary District to limit diversion. It was heard by Special Master Hughes for the Supreme Court of the United States. The court issued a decree in 1930. The decree forces the Metropolitan Sanitary District to increase its program of construction of plants and intercepting sewers so that it would be able to reduce diversion to 1,500 cfs by 1940 for all purposes except water supply.

On June 30, 1967, the State of Illinois adopted water quality standards. These were subsequently approved by the Federal Government. These standards require that canals presently used for industrial cooling water supply, shipping and waste assimilation be upgraded to where they can be used for water supply and recreation.

The B.O.D., suspended solids, and other parameters in municipal treatment plant effluent, industrial waste discharges and combined sewer overflows exceed the assimilation capacity of the canal and river system. None of the major waterways meet the standards.

The adoption of the standards and limitation of diversion waters by decree have forced the Metropolitan Sanitary District to include tertiary treatment at its plants and an equivalent of separation of sewers in combined sewer areas in the future program. The diversion will be further reduced, increasing the severity of the problem, as other communities and sanitary districts request an additional share of the 3,200 cfs for water supply and diversion of sewage effluent from the lake.

The Metropolitan Sanitary District has proposed a $2 billion-dollar, ten-year program to meet the water quality standards and permit higher uses of the waterways.

COMBINED SEWER PROBLEM

In the City of Chicago and older suburbs, there are over 300 square miles (Fig. 2) served by combined sewers. These areas are almost fully developed - having only about 12% vacant property. Many industries using large quantities of water are located in these areas.

Most of the Suburban combined sewer systems discharge their storm overflow to the local streams. These streams flow through Forest Preserves or other recreational areas. Swimming or wading cannot be permitted due to the polluted condition. Fishing is limited to a very few areas.

The greatest number of the 400 overflow points (Fig. 3) from combined sewers discharge directly to the canal system. Use of canal water is generally restricted to cooling water due to the poor water quality.

The cost of separation of sewers would be over $4 billion-dollars. Disruption of the community and loss of business during the construction would add considerable expense to this figure.

Based on present knowledge of storm runoff from urban areas it is doubtful if separation would sufficiently improve the quality of the waterways to meet standards.

FLOOD CONTROL PROBLEM

The Metropolitan Sanitary District has the responsibility of providing outlet capacity for drainage from the Greater Chicago area. The canal system, which discharges through a control structure and power house into the Des Plaines River at Lockport, and the upper portion of the Des Plaines River and its tributaries are the main stormwater outlets.

Rapid urbanization of the area is increasing runoff and peak flows in the waterways. This is illustrated by the fact that the Metropolitan Sanitary District has had to permit the canal system to flow to the lake on eight occasions during severe storms in the last ten years to prevent flooding.

No major improvements to increase the outlet capacity have been made to the Sanitary and Ship Canal or the Des Plaines River since their original construction. Additional capacity in the canal and river system must be provided by deepening and widening, or stormwaters must be detained and gradually released after the peak of the storm. Failure to do this will cause storm flows to be released into Lake Michigan with increasing frequency - a violation of the standards and our ordinances.

Retention of storm flows in surface reservoirs has been a standard flood control practice for many years. The Metropolitan Sanitary District has constructed, has participated in the construction of and has future plans for approximately 15 surface reservoirs on the smaller streams. These are primarily in the separate sewered areas. In the combined sewer area, the flat topography, development of the area, and cost of land limit the number of reservoir sites, therefore, investigation of the potential of subsurface storage becomes desirable. This would include rock tunnels and storage areas.

EQUIVALENT OF SEWER SEPARATION - DEEP TUNNEL PLAN (Fig. 4)

It must be emphasized to meet water quality standards we must collect and treat overflows from the combined sewer area. Conveyance tunnels and storage reservoirs will be required.

Basically, the plan provides for intercepting the combined sewer overflow ahead of the outfall, diverting it into conveyance tunnels and storing the flow in underground and surface reservoirs. It would then be treated at existing and new plants before discharge to the waterway.

Several plans for equivalent of sewer separation have been reviewed. The two basic plans - which are being merged at the present time - are the Deep Tunnel Plan proposed by the Metropolitan Sanitary District, and a composite plan proposed by the City of Chicago (Fig. 5). It has been agreed among engineers in these agencies that underground rock tunnels, combined with storage underground and on the surface, is the most feasible method. Both plans have these features. The City of Chicago has been engaged by the Metropolitan Sanitary District to develop a plan for 11 miles along the North Branch of the Chicago River and the North Shore Channel and prepare plans and specifications for the initial contract. The work in this area can be designed to be compatible with variable plans which may be decided upon downstream. The first tunnels will be constructed within the Niagaran rock formation.

Tunnels paralleling the waterways are proposed which will intercept the overflows. The tunnels will provide storage as well as transport. In addition, large storage reservoirs on the surface and in the rock are proposed in the vicinity of our major sewage treatment works. These storage reservoirs will permit combined sewage to be treated after the storm using peaking capabilities of the plants.

The three rock tunnels presently under construction can be incorporated into the projects. Their point of discharge would be to a conveyance tunnel below them which would greatly increase the discharge capacity due to the additional head available. The construction of these projects is proving the feasibility of using rock moles for tunneling in limestone.

RELIEF INTERCEPTING SEWERS (Fig. 6)

Due to the rapid expansion of the Metropolitan Sanitary District in area, population, residential, commerical and industrial development in the last 15 years, relief sewers are required to convey the waste to our treatment plants. The District doubled in size in 1956. To provide immediate service, future capacity in the then existing intercepting sewers was committed to serving the newly annexed areas. Relief sewers, at a projected cost of $100,000,000 are planned throughout the Metropolitan Sanitary District to provide service when the area is fully developed.

The Metropolitan Sanitary District is presently proposing using the same deep or underflow tunnels, which will convey the combined sewage, as relief sewers to take quantities in excess of the dry weather flow capacity of the existing interceptors. This will eliminate the need of a parallel relief sewer at a higher level to convey the flows to the sewage treatment plants. The tunnels and storage areas will also enable the Metropolitan Sanitary District to reduce or practically eliminate the hourly fluctuation in sewage flows at the plants. By having an even flow through the plant, a better effluent can be maintained, as one of the variables of plant operations is controlled.

Mining storage reservoirs, using multiple headings, is believed to be economical. The underground quarrying of limestone has been done elsewhere in the nation. Cost estimates which we are presently reviewing indicate less than the $5/yard we have in our initial studies. The industry already has developed the equipment required.

TREATMENT FACILITIES

As we will be mining tunnels and storage areas in the rock under our plants, the advantages and disadvantages of constructing treatment facilities underground are being investigated. If the areas for treatment facilities are mined at the same time as the tunnels, the cost would be reduced due to the large cost of setting up a mining operation.

There are advantages in the subsurface which include:

1. Having limited space at the existing plant sites, additional land will be needed to meet the requirement of tertiary treatment and treating combined sewer overflow. Underground construction should minimize the additional land required. Land in the vicinity of the existing plants is very expensive and largely developed.

2. Tests indicate oxygen transfer is speeded up under pressure. By using the available head in a siphon arrangement, it may be possible to cut aeration time thereby reducing the size of facility required and cost of mining below ground.

As we presently believe the only practical method of removing ammonia is to nitrify, we are considering additional batteries of aeration and final tanks to provide two-stage aeration at our major plants. This requires large sites.

We have constructed a small pressurized activated sludge plant. Experiments with this unit, as well as lab experiments, will determine the feasibility of this concept.

PRESENT TUNNELING EXPERIENCES

Tunneling construction methods for water conduits, subways and sewers have been developed in the Chicago area over a period of many years. Mining machines are now extensively used in the clays and granular materials. Rock moles are presently being used on three major contracts in the Chicago area. These contracts are as follows:

...Lawrence Avenue Tunnel, a 13'8" diameter bore, for the City of Chicago, being constructed by S. A. Healy at a cost of $10,792,094. (Fig. 7)

...Calumet Intercepting Sewer 18E, Extension A, a 16'10" diameter bore, being constructed by S&M Constructors for the Metropolitan Sanitary District, at a cost of $6,954,675. The cost per cubic yard excavation, unlined, is $33.50. (Fig. 8)

...Southwest Intercepting Sewer 13A, a 13'10" diameter bore, being constructed by S. A. Healy and Kenny Construction Companies for the Metropolitan Sanitary District, at a cost of $6,210,736. The cost per cubic yard excavation, unlined, is $50.09. (Fig. 9)

These projects are in the Niagaran limestone formation and are located approximately 200 ft. below the surface. They are proving the effectiveness of the rock moles in the Chicago region. The rock is structurally sound, which eliminates the need for a conventional concrete lining for structural support. The contractor is required to grout and stop any leaks which occur. After completion of the mining, the tunnels will be inspected to determine what surface treatment, if any, will be required.

A Jarva machine was used on the Calumet 18E Ext. A sewer. The mining is nearly complete. The average progress per hour was 4.9 ft. A Robbins mining machine was used on the Southwest 13A Contract. The mining is presently completed. (Fig. #10)

Infiltration enters the project from the shafts, line holes, some of the horizontal bedding planes and vertical faults. No grouting was performed during construction on the Calumet 18E Ext. A job. Portions of the Southwest 13A project were grouted before and during the mining operation. After the mining of these tunnels, additional grouting is required.

These tunnels are designed as siphons which will discharge to the waterway during all but the smaller storms. They will be de-watered by a pumping station after the storm into an intercepting sewer, which will convey the flow to a major sewage treatment plant. The storage in the system is used in this manner to reduce the pollutants discharged to the stream.

In our area experience with rock moles is limited to about 3 years. Improvements are continually being made. It is expected that the next generation of machines will greatly improve operations and procedures.

GEOLOGY OF REGION

The upper strata consist mostly of tills, lacustrine clays with some stratified deposits and sand dunes (Fig. #11). The Niagaran and Galena Platteville limestone formations underlie the urban area - separated by a shale formation - the Niagaran being approximately 0 to 300 ft. below the surface. To date, except for quarrying from the surface in the Niagaran and construction of water tunnels and a small number of wells, little use has been made of the limestone formations.

In the last few years, over $1,925,000 have been expended on investigations which included seismic, drill holes and rock cores, and logging existing and new wells. Additional information will be obtained as new tunnel alignments are decided upon. The data have provided substantial information on the rock strata.

SUMMARY

Space is at a premium in metropolitan areas. Surface reservior sites for flood and pollution control are limited in number and costly. Where subsurface projects can be constructed at prices competitive with more conventional surface solutions and having additional benefits, they should be carefully evaluated.

Underground construction provides advantages that should be considered in urban areas.

1. It greatly reduces the disruption of an urban community. Utilities are generally not affected as they are with open-cut work. The costs of moving utilities are generally assumed by the utility company. This is a cost which is not reflected when open-cut and tunneling estimates are compared. There is little inconvenience to residents compared with open-cut work. The construction does not interfere with the operation of existing commercial and industrial establishments.

2. As a part of a system, the deep tunnel can provide an equivalent of sewer separation. This will improve water quality and enable the area to meet established water quality criteria. The cost should be less than 25% of the cost of sewer separation. Due to the pollution in storm runoff from urban areas, sewer separation would not sufficiently improve the quality in the waterways to meet the established standards. The system could be enlarged to handle polluted runoff from separate sewer areas.

3. Providing an outlet far below the existing local sewer systems will improve their performance and permit much more flexibility and economy in the design of relief systems in the local communities.

4. Collecting, storing and treating combined sewage before releasing to the canal will reduce the amount of diversion water required to meet the standards. This is of great importance as the demand for lake water in Northeastern Illinois increases.

5. The large volumes of underground storage provide a means of reducing peak flows in the channel system - eliminating the need for discharge of contaminated water to Lake Michigan in severe storms as has been required in the past. It should reduce or eliminate the need for major widening and deepening of the waterways.

6. The mined rock from many of these projects can be stored in existing quarries for future commercial sale, used to create additional park land or in the lake airport which is under consideration, used to create a recreational mountain in this flat region, etc. Many of these uses provide a benefit that can be measured in dollars - reducing the cost of the projects.

7. Surface and underground reserviors can permit regulation of flows to the treatment plants - greatly improving their operational efficiencies.

8. Underground construction can permit greater expansion of plant facilities at existing sites.

9 Improvements in design in tunneling equipment are going to further reduce the price of excavation.

QUESTION: How will solids be handled in the tunnels or room and pillar type storage?

NEIL: We felt that as a part of the design of the deep tunnel system we would need some settling tanks for the removal of solids. We do not have the complete answer as yet, but we feel it would be easier to collect solids prior to entering the tunnel. The requirements for aeration and the length of holding time in the underground storage areas are being studied.

QUESTION: What are the differences between the underflow storage plan, the deep tunnel plan and the Chicago drainage plan?

NEIL: The basic differences between the deep tunnel and underflow plans is the amount of storage. The deep tunnel plan has about 60,000 acre feet to collect all overflow, the underflow plan has about 20,000 acre feet. The drainage plan involves treatment at 400 overflow points and increasing the capacity of the canal system.

QUESTION: How many miles of tunnels do you envision in the final system and how many are now under design for the next phase?

NEIL: Currently eleven miles are planned. The total tunnel system for Chicago only would be from 40 to 50 miles. Including the suburbs the system would be in the hundreds of miles.

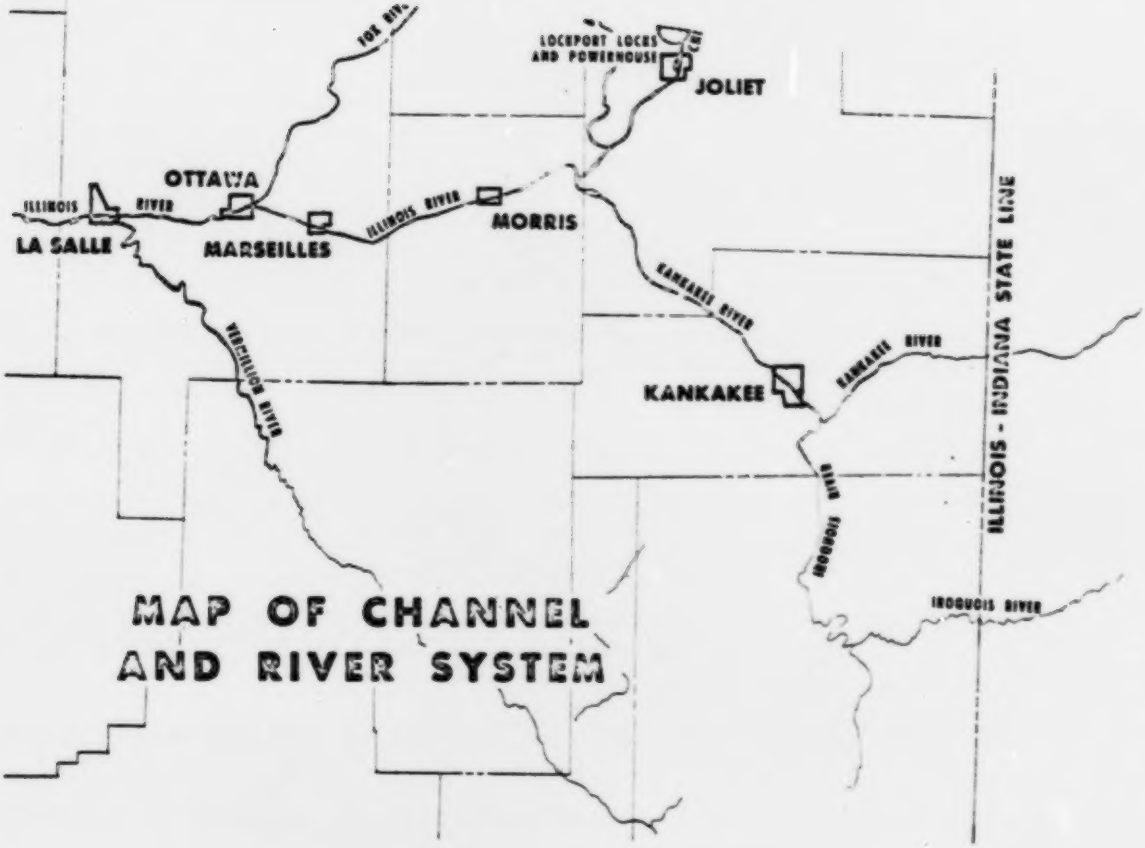
QUESTION: What techniques will be used for constructing the shafts?

NEIL: A conventional mining technique has been used. Future contracts may work either from the top down or the bottom up; this will be left as an option.

ROSENKRANZ: The hydraulics of the down shafts are critical and are being studied. The FWQA has initiated some work at the University of Minnesota.

QUESTION: What are the legal implications of tunneling under private property?

NEIL: We try to stay under public streets, otherwise we obtain an easement with the owner.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KANKAKEE RIVER
KANKAKEE
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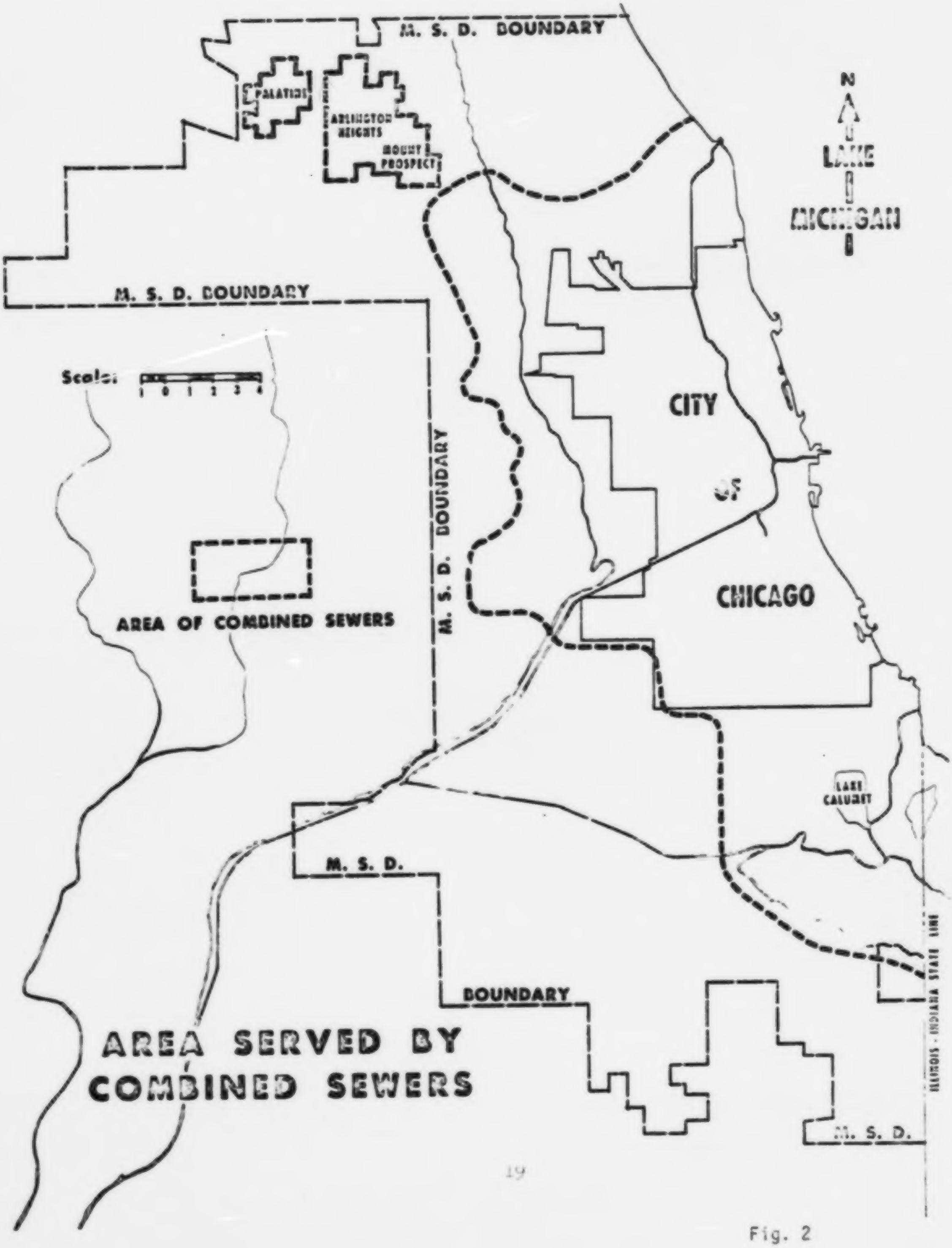
IROQUOIS RIVER
IROQUOIS RIVER
ILLINOIS - INDIANA STATE LINE
MAP OF CHANNEL AND RIVER SYSTEM

Fig. 1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AREA OF COMBINED SEWERS
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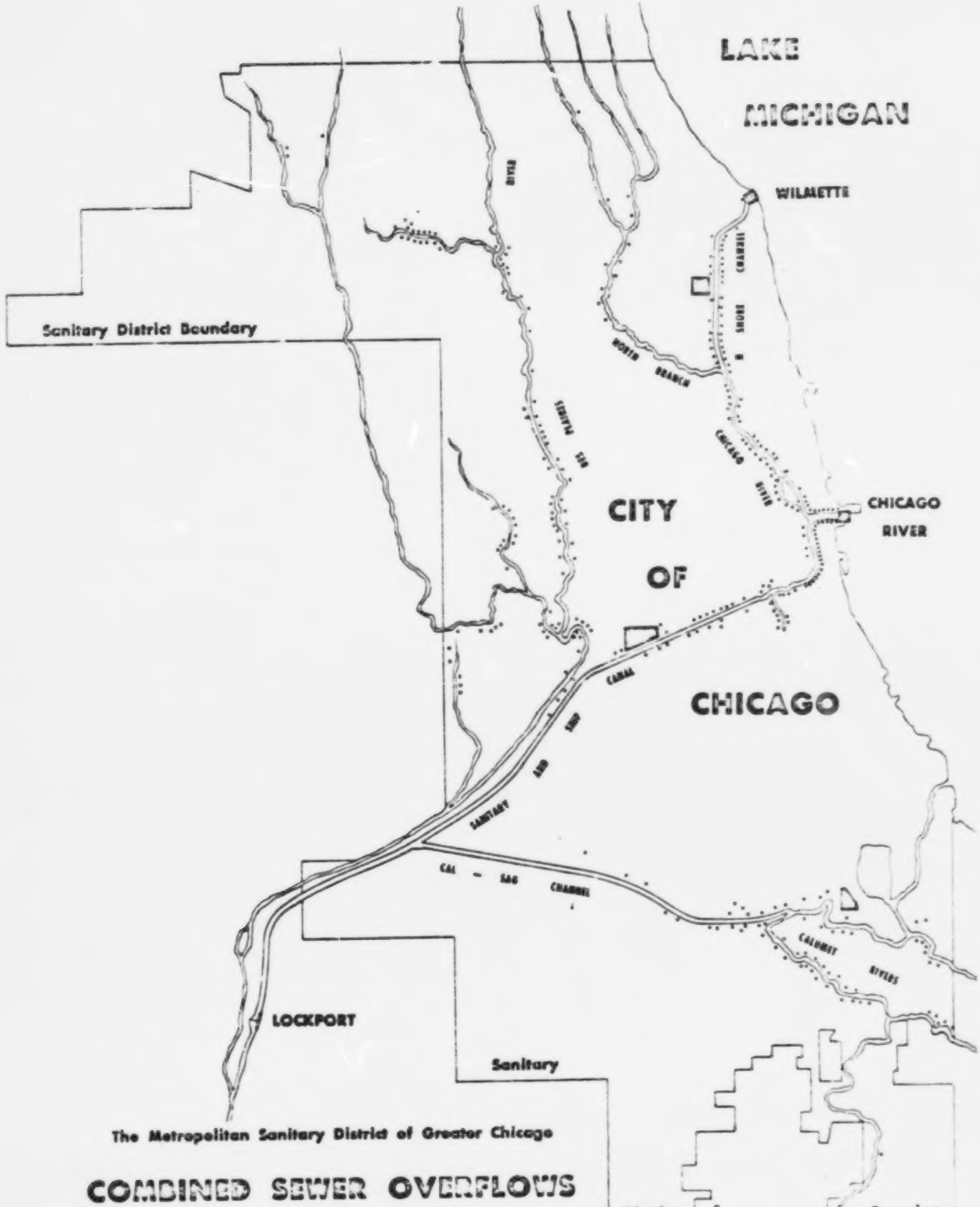
ILLINOIS - INDIANA STATE LINE

AREA SERVED BY COMBINED SEWERS

Fig. 2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DES PLAINES
NORTH SHORE CHANNEL
NORTH BRANCH
CHICAGO RIVER
CITY OF CHICAGO
CHICAGO RIVER
SANITARY AND SHIP CANAL
CAL - SAG CHANNEL
CALUMET RIVERS
LOCKPORT
Sanitary District Boundary

The Metropolitan Sanitary District of Greater Chicago

COMBINED SEWER OVERFLOWS

• INDICATES OVERFLOW TO CANAL OR TRIBUTARY

Fig. 3

DEEP TUNNEL PLAN
FOR
FLOOD & POLLUTION CONTROL

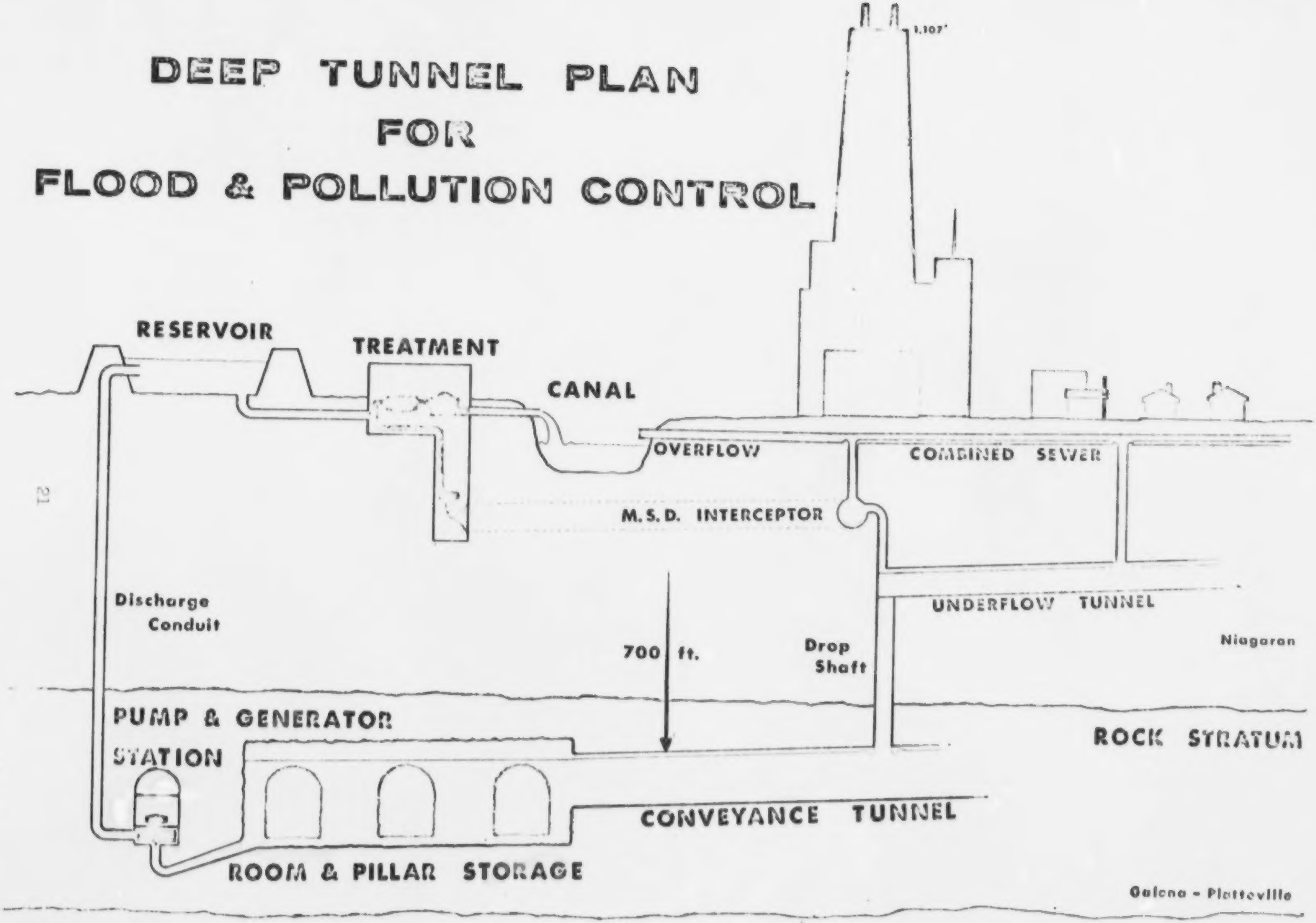
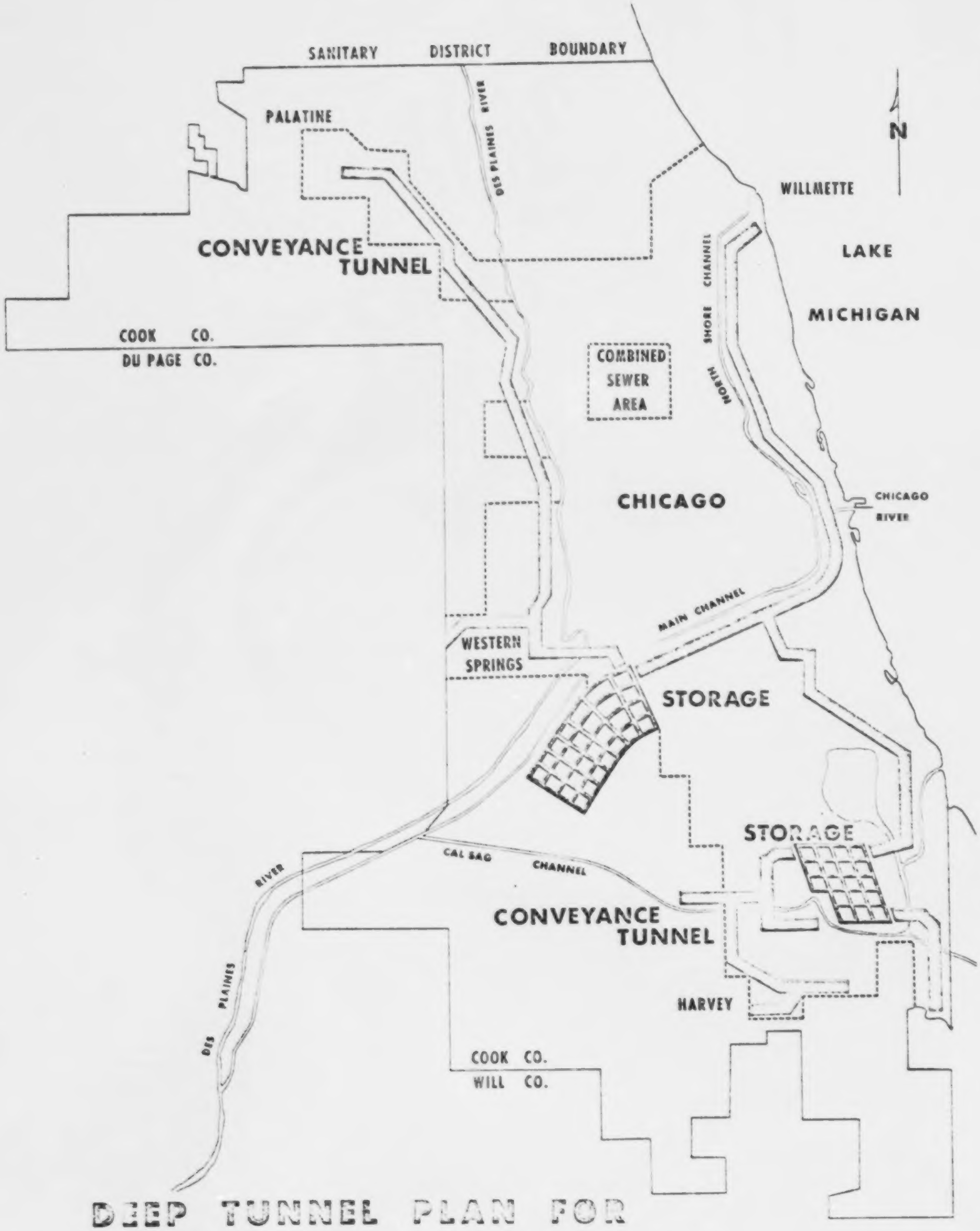

1,107'
RESERVOIR
TREATMENT
CANAL
OVERFLOW
COMBINED SEWER
M.S.D. INTERCEPTOR
Discharge Conduit
UNDERFLOW TUNNEL
Niagaran
700 ft.
Drop Shaft
PUMP & GENERATOR STATION
ROCK STRATUM
CONVEYANCE TUNNEL
ROOM & PILLAR STORAGE
Galena - Platteville

Fig. 4


SANITARY DISTRICT BOUNDARY
N
PALATINE
DES PLAINES RIVER
WILLMETTE
CONVEYANCE TUNNEL
LAKE MICHIGAN
NORTH SHORE CHANNEL
COOK CO.
DU PAGE CO.
COMBINED SEWER AREA
CHICAGO
CHICAGO RIVER
MAIN CHANNEL
WESTERN SPRINGS
STORAGE
STORAGE
CAL SAG CHANNEL
RIVER
CONVEYANCE TUNNEL
DES PLAINES
HARVEY
COOK CO.
WILL CO.

DEEP TUNNEL PLAN FOR
FLOOD AND POLLUTION CONTROL #2

Fig. 5

Reproduced from best available copy.

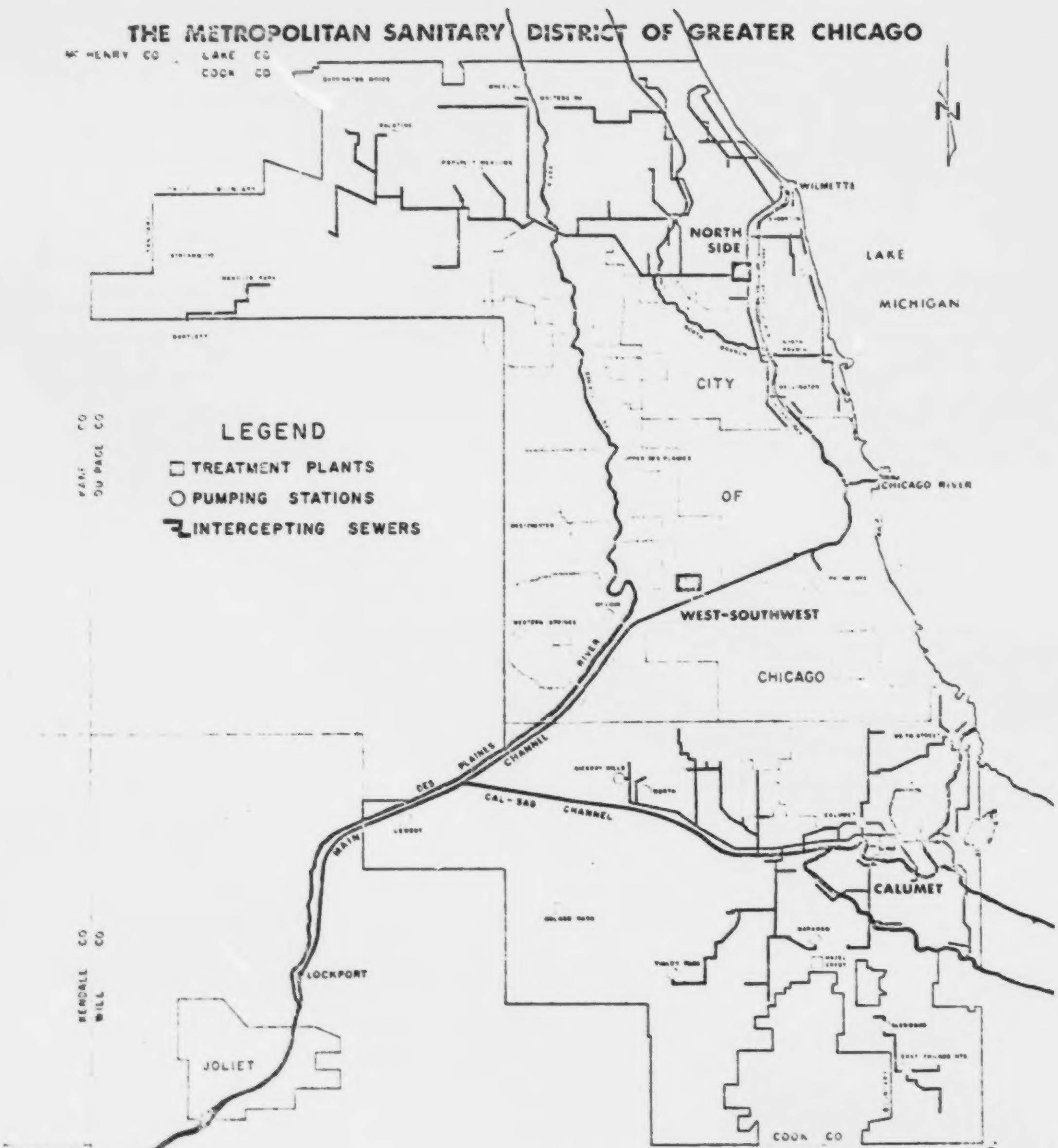

THE METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO
McHENRY CO
LAKE CO
COOK CO
WILMETTE
NORTH SIDE
LAKE
MICHIGAN
CITY
OF
CHICAGO
CHICAGO RIVER
WEST-SOUTHWEST
LEGEND
TREATMENT PLANTS
PUMPING STATIONS
INTERCEPTING SEWERS
KANE CO
DU PAGE CO
DES PLAINES RIVER
MAIN CHANNEL
CAL-SAG CHANNEL
CALUMET
LOCKPORT
JOLIET
KENDALL CO
WILL CO
GRUNDY CO
COOK CO
WILL CO
ILL.
IND.

Fig. 6

ROCK TUNNEL SUMMARIES

LAWRENCE AVE. SEWER SYSTEM
CONTRACT NO. 1

Length of Tunnel:	
In Lawrence Avenue.	* 9,126 feet of 15'6½" x 19'5"
In Lawrence Avenue	12,670 feet of 12 foot dia.
In Harding Avenue	3,968 feet of 12 foot dia.
	25,764 feet
	*6,760 feet mined by machine to 13'8" and enlarged by drill and blast method and 2,366 feet full face drill and blast with finished section of 8" liner to dimensions of 15'6½" x 19'5"
Depth below Ground	245 feet max, 220 feet min
Slope of Sewer	2.5 per 1000
O.S. Diameter specified: (Mined by Machine):	
In Lawrence Ave. (1st 9,126)	18'4" dia.
In Lawrence Ave. (last 12,670')	13'4" dia.
In Harding Ave. (3,968')	13'4" dia.
O.S. Diameter Actual:	
In Lawrence Ave. (1st 9,126')	16'10½" x 20'9" D&B or enlarged from machine bore of 13'8"
In Lawrence Ave. (2nd 12,670')	13'9" dia.
In Harding Ave.	13'9" dia.
I.S. Diameter:	
In Lawrence Ave. (1st 9,126')	15'6½" x 19'5" (lined)
In Lawrence Ave. (2nd 12,670')	12'0" dia. (if lined)
In Harding Ave. (3,968')	12'0" dia. (if lined)
Tail Tunnel	61 feet
Shaft	27 feet dia. and 256 feet deep
Contract Costs (Bid):	
1. Shaft	600,000
2. 12 foot dia. Tunnel	4,658,640
17 foot dia. Tunnel	3,732,534
3. 12 foot dia. Lining	993,280
17 foot dia. Lining	730,080
4. Rock Bolts	67,500
5. Wire Mesh	5,000
TOTAL	$10,792,094

Fig. 7

ROCK TUNNEL SUMMARIES

LAWRENCE AVENUE SEWER SYSTEM
CONTRACT NUMBER 1

Increased Storage (without conc. lining in the 12' dia. sections)	31% Increase in Volume
In Lawrence Ave. (west of Sta. 91+50)	16,610 cubic yards
In Harding Ave.	5,202 cubic yards
TOTAL	21,812 cubic yards
Award Date	November 1, 1967
Term of Contract	1,095 days
Specified date of completion	November 5, 1970
Normal Shifts	24 Hours Monday through Saturday
Total Progress to date:	13,094 feet
Lawrence Avenue:	
Machined mined (13'-8")	9,126 feet 9/30/70
Drill and Blast Enlargement	9,126 feet 9/30/70
In Harding Avenue (13'9")	3,968 feet 9/30/70
Progress Max. Week	347 feet 2 Shifts
Progress Max. Day	92 feet (2 Shifts)
Maximum Penetration ft./hr.	8.7 maximum
Comp. Strength Rock p.s.i.	11,400 to 29,600
Mining Machine:	
Manufactured by	Lawrence Mfg. Company
Thrust of Machine	1,300,000 lb. (Max.). 850,000 lb. oper.
Drive of Machine	5-125 hp. motors
Operation Voltage	480 Volts
Make of Bits	Lawrence Mfg. Company
Number of Cutters	29 Disc-Type with carbide inserts
Diameter of Cutterhead:	
Machine Number 1	13'-8" dia. in Lawrence Avenue
Machine Number 2	13'-9" dia. in Harding and Lawrence
Length of Machine:	

ROCK TUNNEL SUMMARIES

LAWRENCE AVENUE SEWER SYSTEM
CONTRACT NUMBER 1

Assembly Drawbar Power Train Auxiliary Power, Train TOTAL	19'-11" 15'-11" 23'-7" 25'-4" 84'- 9"
Tunnel Power Line	4,160 Volts
Conveyor System Manufacturer	Lawrence Mfg. Co. with a Goodyear Belt 24" wide by 84' long
Muck Cars	6 Cubic Yards
Length of Train	9 Cars
Track Gauge	36"
Locomotives	10 Ton, Plymouth Diesel, 86 hp.
Ventilation	28" Vent Line 2-40 hp Vent fans made by the Joy-Axivane Co. 14,000 CFM each. One 15 hp. fan at street level to prevent any line back pressures.
Contractor	J. McHugh Construction Co. S. A. Healy Co., and Kenny Construction Co. (a joint venture)
esident Engineer	John Redmore

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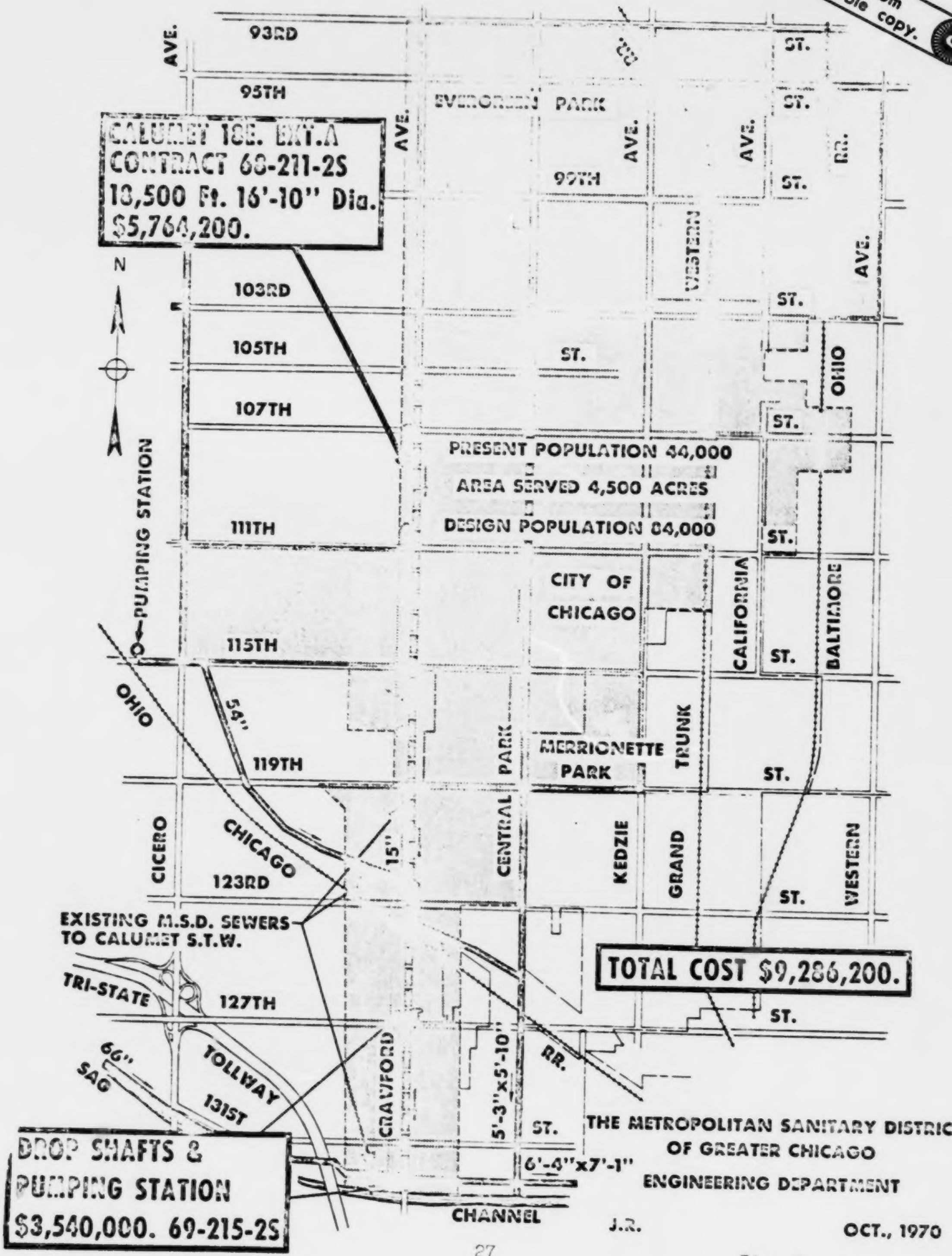
CALUMET 18E. EXT.A
CONTRACT 68-211-2S
18,500 Ft. 16'-10" Dia.
$5,764,200.
93RD
95TH
99TH
103RD
105TH
107TH
111TH
115TH
119TH
123RD
127TH
131ST
AVE.
ST.
RR.
EVERGREEN PARK
WESTERN
OHIO
CALIFORNIA
BALTIMORE
N
PUMPING STATION
PRESENT POPULATION 44,000
AREA SERVED 4,500 ACRES
DESIGN POPULATION 84,000
CITY OF CHICAGO
54"
15"
OHIO
CHICAGO
CICERO
CENTRAL PARK
MERRIONETTE PARK
KEDZIE
GRAND TRUNK
EXISTING M.S.D. SEWERS TO CALUMET S.T.W.
TOTAL COST $9,286,200.
TRI-STATE
TOLLWAY
66"
SAG
CRAWFORD
5'-3"x5'-10"
6'-4"x7'-1"
CHANNEL
DROP SHAFTS &
PUMPING STATION
$3,540,000. 69-215-2S
THE METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO
ENGINEERING DEPARTMENT
J.R.
OCT., 1970

Fig. 8

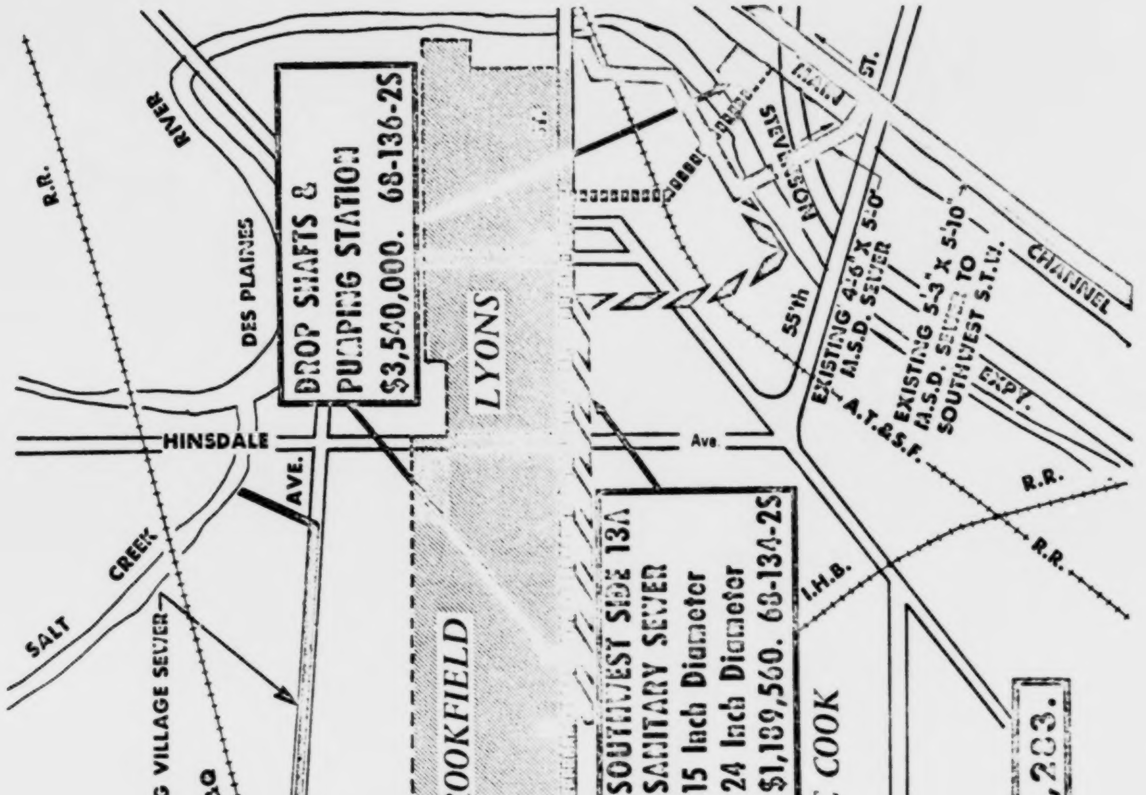
DROP SHAFTS & PUMPING STATION
$3,540,000. 68-136-2S
SOUTHWEST SIDE 13A SANITARY SEWER
15 Inch Diameter
24 Inch Diameter
$1,189,560. 68-134-2S
SOUTHWEST SIDE 13-A
3.4 Miles of 13'-10" Dia.
$5,350,723. 67-112-2S
TOTAL COST $10,080,283.
AREA SERVED
1,900 ACRES
20,000 Existing Population
34,180 Projected Population
DES PLAINES RIVER
R.R.
SALT CREEK
EXISTING VILLAGE SEWER
CB&Q
HINSDALE Ave.
AVE.
EBERLY AVE.
GILBERT AVE.
OGDEN
COSSITT AVE.
LAGRANGE
BROOKFIELD
LYONS
MC COOK
47th
55th
PLAINFIELD RD.
I.H.B.
A.T.&S.F. R.R.
R.R.
EXISTING 4'-6" X 5'-0" M.S.D. SEWER
EXISTING 5'-3" X 5'-10" M.S.D. SEWER TO SOUTHWEST S.T.W.
SOUTHWEST EXPY.
CHANNEL
ST.
METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO
ENGINEERING DEPARTMENT
A.G.B.
OCT., 1970

Fig. 9

ROCK TUNNEL SUMMARIES

	Lawndale Avenue & 47 St. SWIS 13A		127th & Crawford Ave. Calumet 18E-Ext. A	
Length of tunnel	17,553 feet		18,320 feet	
Depth below ground	235 Max. 201 Min.		223 Max. 216 Min.	
Slope of Sewer	2.1 per 1000		1.5 per 1000	
O.S. Diameter specified	13' - 4"		16' - 4"	
O.S. Diameter Actual	13' - 10"		16' - 10"	
I.S. Diameter (if lined)	12' - 0"		15' - 0"	
Tail Tunnel	250 feet		260 feet	
Shaft	30' and 28'x206' deep		29' and 27'x223' deep	
Contract Costs	Bid	Revised	Bid	Revised
1. Shaft	850,000	850,000	1,000,000	1,000,000
2. Tunnel	4,567,206	4,503,723.60*	4,763,200	4,763,200
3. Lining	793,530	0	1,190,476	0
4. Bulkhead	included above		1,000	1,000
TOTAL	6,210,736	5,350,723.60*	6,954,675	5,764,200
	*includes credit for rock material and refund on Elec. Agreement ($3.60/lin. ft. credit on rock).			
Increased Storage (without Conc. lining)	36% increase in volume 24,000 Cubic Yards		26% increase in volume 31,000 Cubic Yards	
Award Date	June 6, 1968		May 17, 1968	
Term of Contract	930 days		933 days	
Specified date of completion	January 5, 1971		December 16, 1970	
Normal Shifts	24 hrs. Mon. through Friday		24 hors Mon. through Saturday	
Progress to date	17,553 (9/24/70)		16,018 (9/29/70)	
Progress Max. Week	591 feet		607 feet	
ogress Max. Day	144 feet		129 feet	

Fig. 10

	Lawndale Avenue & 47th St. SWIS 13A	127th & Crawford Ave. Calumet 18E-Ext. A
Maximum Penetration ft./hr.	5.5 avg. 8.2 maximum	7.6 maximum
Comp. Strength Rock psi.	15,000 to 24,900	23,500 to 39,000
Mining Machine		
Manufactured by	James S. Robbins & Assoc. Inc.	Jarva Inc.
Thrust of Machine	890,000 lb. (max.)	2,200,000 lb. (max.)
Drive of Machine	6-100 hp motors	8-125 hp motors
Operation Voltage	460 Volts	480 Volts
Make of Bits	James S. Robbins & Assoc.	Reed Drilling Tools
Number of Cutters	27 Disc-Type plus Tri-Cone	54 Reed Type Q.K.C.
Length of Machine	37 feet	35 feet
Dia. of Cutterhead	13 feet 10 inches	16 feet 10 inches
Conveyor System manufactured by	Moran Engineering Co. 96' bridge conveyor (20" widebelt) to 132' (18" wide belt) car loader	Card Corporation 260 feet conveyor supporting a 30" belt
Muck Cars	4.4 Cubic Yards	10 cubic Yards
Length of Train	10 cars	10 cars
Track Gauge	24"	36"
Locomotives	10 Ton, Plymouth Diesel, 70 hp	15 Ton, Plymouth Diesel, 160 hp
Ventilation	30" Vent line 2-100 hp Vent fans @12,000 CFM each	36" Vent line Joy-Axivane fans 31,000 CFM max
Contractor	S. A. Healy Company and Kenny Construction Co. (a joint venture)	S & M Contractors Inc.
Resident Engineer	George A. Taylor	Thomas P. Vitulli

Session 2

MULTIPLE PURPOSE BENEFITS OF DEEP STORAGE AND TUNNELING

Moderator - G. Rohlich

Section 3

THE ROLE OF STORAGE IN ECONOMICS OF SEWAGE TREATMENT PLANT DESIGN

by

William J. Bauer
President
Bauer Engineering, Inc.
20 North Wacker Drive
Chicago, Illinois 60606

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THE ROLE OF STORAGE IN THE ECONOMICS OF SEWAGE TREATMENT PLANT DESIGN

I should like to speak about a very simple concept, discuss some of the ramifications of the concept, and then attempt to answer any questions you may have. The basic concept is to secure a higher load factor or higher per cent utilization of treatment plant facilities. It has its parallel in the pumped storage idea applied to an electric utility. The variation in electric power demand is quite marked, so of course is the demand for treatment plant capacity. Now the significance of this variation is becoming greater with time, because the cost of treatment--that is, the number of dollars it takes to buy a million gallons per day capacity--is going up. We require more and more of these treatment plants, and we require less and less pollution to be discharged from them. So, if there's any way of making this expensive plant work harder, it seems that we should take a look at it. Obviously, the way to do this is with storage, and the economic choice involves the relative costs of water storage and treatment capacity. I shall proceed first with some of the graphics that I have, then I shall present some of the results of a study we made on this matter.

FIG. 1

These two curves illustrate the point that I was making about the relative costs of storage and treatment capacity. Here you see one curve labelled "higher flow rate" and the other "lower flow rate," which would be associated with a higher load factor. This is merely a schematic representation of the fact that with primary treatment only the difference in cost was small so that it was not as important to regulate flow through the plant. As we begin to spend more money in reducing the permissible pollution the spread between these curves becomes greater. There is getting to be a much larger cost differential as we are requiring more and more advanced forms of treatment.

FIG. 2

The scale on the left is the BOD discharged in the effluent from the Southwest Sewage Treatment Plant of the Metropolitan Sanitary District of Greater Chicago. This diagram pertains to a one-year study made by the Federal Water Pollution Control Administration. The scale at the bottom is per cent of time. Three curves are shown: The lower one is for typical average flows. Each one of these curves was plotted with the aid of daily laboratory data taken from the analysis of the effluent of the treatment plant. If you read the chart at a certain "per cent of time" on a vertical line, you will find that a higher flow going through the plant has, in general, a higher BOD in the effluent. So, at any vertical line we wish to examine, we find that superior treatment is achieved with the lower flow rate, as one would expect.

FIG. 3

Here is an actual plot of data taken from a treatment plant in the suburban Chicago area; you can see quite a variation with pronounced peaks and valleys. A very sharp peak corresponds to some rain. I have drawn two lines: one line represents the dry weather flow, and the other represents 1.33 times the dry weather flow. We were examining in this study, on the basis of the actual historical variation in these flows, what volume of storage was required in order to limit the flow rate through the treatment plant to this flow of 1.33 times the dry weather flow. Some sewers have many direct connections which allow storm water to get into them, producing a very wide ranging fluctuation.

FIG. 4

This plot is taken from still another example in the Chicago suburban area. It is based on historical variation of flows in a two-year period at Bloom Township Sanitary District. Plotted vertically is the treatment capacity in millions of gallons per day, and across the bottom the storage that would be required to not exceed that flow capacity. This is of course the end result of the type of statistical study I described when discussing the preceding diagram. We analyzed two different years: One curve represents the relationship for the '67 flows, and the upper one represents the '68 flows. The trend line shows that we could achieve the same end result with the capacity of 15 million gallons per day if we could store 7 million gallons as we could with a 20 mgd plant with no storage at all. The slope of this curve is obviously very important and it is something that depends on the statistical variation in the particular flow system. Now, if this curve is steep it is obvious that a great deal is gained by storage; conversely, if it is flat, one does not have very much incentive to store.

FIG. 5

These curves were developed for the Deep Tunnel Plan study for the Metropolitan Sanitary District of Greater Chicago. Again, you see on this side a measure of the amount of pollution (in this case it is the volume of spill). Running across the bottom again is storage. As we increase the storage of the system, we are able to reduce the frequency of spill so that the total quantity of the spill is also reduced. We have a family of curves here, each for a particular sustained rate of flow through the treatment plant. The distance between two curves at any particular level is the incremental storage required to achieve the equivalent treatment in terms of the same volume of water spilled into the waterway. The variation here shows, for instance, that storage would permit us to reduce the treatment capacity from 1500 to 375 mgd. That means that the treatment capacity could be reduced to 1/4 of what otherwise would be required in order to handle a given situation. I should like to contrast this diagram with the preceding one because the previous one was presumed to be a separate sewer system--one not supposed to have any storm water connections to it--and this one is a combined system. You notice a great deal more benefit from using storage in connection with the combined system. The effect of storage on treatment plant capacity depends a great deal on whether the system is separate or combined.

FIG. 6

Now there is also a question of the effect of storage on the cost of transporting the water. If the collection system is rather long, one achieves a relatively greater economic advantage from a high per cent utilization of that transportation system. This diagram is a schematic showing (in dotted lines) a flow-regulating storage system at the head of a collector system, then at the treatment plant itself an additional storage to perform the function of even further regulating the flows through the treatment plant. The lower diagram illustrates the typical variation during a day for domestic flows giving one way of evaluating, on a daily basis, the effect of storage on regulating flow through the plant.

I should like to cite some figures on the relative costs of these two systems: First of all, with regard to the cost of storage--this can vary widely, depending upon the facilities that are provided. In the Deep Tunnel Plan studies of the Metropolitan Sanitary District of Greater Chicago the bulk storage which was proposed in the mined portions of the plant carried a price tag of about $5 per cubic yard. That did not include the cost of handling solids, and the cost of aerating the flows that would be stored there temporarily. If you divided the $5 (call it $5.40) by 27 cubic feet in a cubic yard, the result is about $.20 per cubic foot of space. By the time one adds facilities for handling solids, the cost will increase to perhaps $1, or conceivably, $1.50 per cubic foot of storage. On the other hand, the cost of concrete boxes with lids at ground levels, scraper mechanisms for handling solids, plus some aeration facilities would probably be in the vicinity of $5 a cubic foot. The range we would expect to talk about is in the neighborhood of $1.50 to $5.00 per cubic foot, or about $40 to $135 per cubic yard, or $64,000 per acre foot, or more. Most of the cost in the case of the underground system is tied up in the facilities for handling solids and aerating.

I should like to give you some analyses here of relative costs of a 30 million gallon per day plant, with and without storage. I have used one unit cost for the treatment facilities themselves and that was $400,000 per mgd capacity. We also assume that with no storage at all, we could provide two times dry weather flow for the basic treatment plant capacity, meaning that it would have a capacity of 60 million gallons per day. Now the 60 million gpd multiplied by the $400,000 per million gallons gives $24 million for the treatment plant capacity without storage. I recognize that quite commonly activated sludge plants are designed for little more than 1.0 times dry weather flow as far as the system of aeration, the aeration chambers, and the biological process are concerned, but, that they are designed for a hydraulic capacity of 2 or 2-1/2 times the dry weather flow. This procedure is based on an acceptance of a reduced level of treatment during the time in which the plant is overloaded. Now, one of the basic assumptions I am making in these analyses is that this kind of operation is not going to be permissible for very much longer, because it is in the same category as dumping or bypassing when the plant is down. So, in assigning the unit costs, I have said that a plant averaging 30 million gpd could be designed to perform well at a peak rate of 60. That is the background for the $24 million figure.

Then I examined a unit cost of $45,000 per acre foot of storage capacity and added varying amounts of storage capacity and calculated the corresponding reduction in treatment capacity. With 13 million gallons of storage capacity, the plant cost was reduced by $4 million. The cost of that storage was $1.8 million, so that there was a net saving of $2.2 million. Going still further, reducing the treatment plant capacity to the level corresponding to 26 million gallons of storage, the treatment plant was estimated to be $16 million, and the storage about $3.6 million, or a total of $19.6 million compared to the $24 million for the plant without any storage. These figures, of course, depend entirely on the statistical relationship between the volume of storage and the corresponding possible reduction in treatment plant capacity. Obviously, it also depends upon the unit cost of storage. If the storage is underground (as would be a convenient location in a crowded urban area like Chicago) one would have an added cost of pumping water back up. An analyses of the amount of that storm water that would be passed to a lower storage zone, then pumped back up to the surface during a typical year in Chicago, resulted in about 14% ratio of storm water volume to total water volume.

There are some advantages to providing this storage other than possibly an economic one. If the rate of flow through the plant is capable of being controlled, the processes can be adjusted to a better degree so that the performance is improved. The flow into the receiving stream is better regulated, so that there is a higher sustained flow and fewer high peaks. In the event there is a mechanical breakdown in the plant, the storage it provides is an optional one to dump the water which otherwise might be directly bypassed into the stream. We did not make a statistical study to obtain a correlation between the occurrence of these natural breakdowns and the occurrence of the storms; but, I think it is evident that one would not expect these events to happen at the same time, therefore, the storage could be used at least in part to catch the mistakes. The problem of the overflows from combined sewers, of course, is one which is attacked directly by this method of providing storage at the treatment plant.

The matter of duration of storage is important, because long periods of storage would require facilities for keeping the water aerobic. We found that the duration of storage is typically a matter of several days, which would require aeration. Because of the large fluctuation in the depth of water in the storage zone (particularly if it is going to be underground storage) we felt that there should be new methods for aerating these deep flows. One of the ideas proposed is a floating aerator which goes up and down with the water surface which has a telescoping draft tube on it so that the entire depth of the flow is (or depth of the water stored) affected by the currents produced by the floating aerator. The solids that would be deposited in these locations would require handling. We envisage separate solids handling facilities and separate pumps to receive the material from a bottom scraping mechanism.

This concept is basically a very simple one. It has not been applied to sewage treatment plants in the past, I believe, largely because there was no economic incentive to do so. The economic and performance factors are changing; we are spending a great deal more per million gallons per day capacity now as we get into more and more advanced forms of treatment. The rules of the game have also changed, and now we're talking not about what our average percent removal is, but we're talking about the impacts on the receiving stream--whether those impacts are of short duration or of long duration.

I believe that we are going to be seeing more and more of the application of this storage function to the treatment system and I think it will be found in two locations--in the treatment plant, and (in the case of long transportation systems) out at the end of the line in order to give higher percent utilization of the sewer itself. This type of design can very well be incorporated in new plants and old sewers in order to gain higher utilization of those existing facilities.

QUESTION: Would you reduce the size of the lateral sewers because you reduce the size of the treatment capacity?

BAUER: I would talk about two flow rates: one would be the flow rate at which you would move water away from the places where it originated to the treatment plant, and the other would be the flow rate through the treatment plant itself. You could have a high flow rate to the storage and a lower flow rate from the storage to the treatment. Now, if I understand your question--your saying: why don't you store it out in the neighborhoods? Yes--and, if you did put the storage out there you would gain a twofold benefit: you would not only reduce your treatment plant capacity, but you would reduce your transportation cost. The only limitation that I can see to that is the practical limitation of size, because, if you are going to have a good storage facility it has to be large enough to have the solids handling and the aeration facilities with it, and warrant some kind of attention by personnel so that you do not want them all over the place, but at some strategic location. I believe it would be very practical to do this in a large system.

QUESTION: What is the storage volume of the sewers themselves?

BAUER: That is a very important factor. I know that in the City of Detroit this has been one of their considerations for a long time in sizing a sewer; to evaluate its effect on the treatment capacity. The Racine Avenue Pumping Station (in Chicago) has very large sewers coming to it and, as I recall, they amount to something like a quarter of an inch or so, a fairly substantial amount of storage, maybe a quarter of an inch spread over the watershed. The minimum that we are thinking about for the Deep Tunnel plan I believe is 1 in.; so, you see that existing storage in some of these large interceptors can be a very substantial fraction of that. In the design of the system that certainly has to be taken into account. I am talking storage, in addition to the storage that you would have anyway, because of the volume of the system.

QUESTION: What are the costs of pumping the water to the surface?

BAUER: Those costs were included in the cost of storage; we capitalized those costs. I think we took the annual cost and multiplied by 15 in order to get the equivalent capital cost. As I say, the $1.50 included an allowance for the pumping from the lower elevation.

QUESTION: What about the $5?

BAUER: That $5 figure corresponds to a concrete box constructed near the surface and includes a small allowance for pumping.

QUESTION: What about the effect upon the activated sludge process because of bringing in the stored sewage?

BAUER: We are talking about 1.25 ratio of total flow to average flow so that the capacity of the treatment would be 1.25 times dry weather flow, assuming we had all the storage required. We would be adding sewage that had already been partially treated as it would be aerated down below. There would be a drop-off in the BOD. This would mean that some of the food for bacteria would have already been used up and would not be available for the bacteria that were operating in the activated sludge plant. However, it may be that the amount of variation in food supply with storage would be less than without storage. We have no way of proving that and I think it is something that could be studied, it would be very interesting. Such a study would actually stimulate the aging of the sewage and add it to fresh sewage to see what effect it had on treatment. Implicit in my studies to date is the assumption that there would be no fundamental change because of the addition of the aged stored sewage.

QUESTION: Would there be a different strain of bacteria developed in the Deep Tunnel and could these be consistently introduced into the treatment plant? What would be the effect on the treatment process?

BAUER: This has not been evaluated so I do not have any way of answering your question.

QUESTION: Have you related the savings in treatment plant costs for the combined sewer area that the Sanitary District serves to this figure (figure $600 million to one billion for various ramifications of the Deep Tunnel and Underflow Plan)?

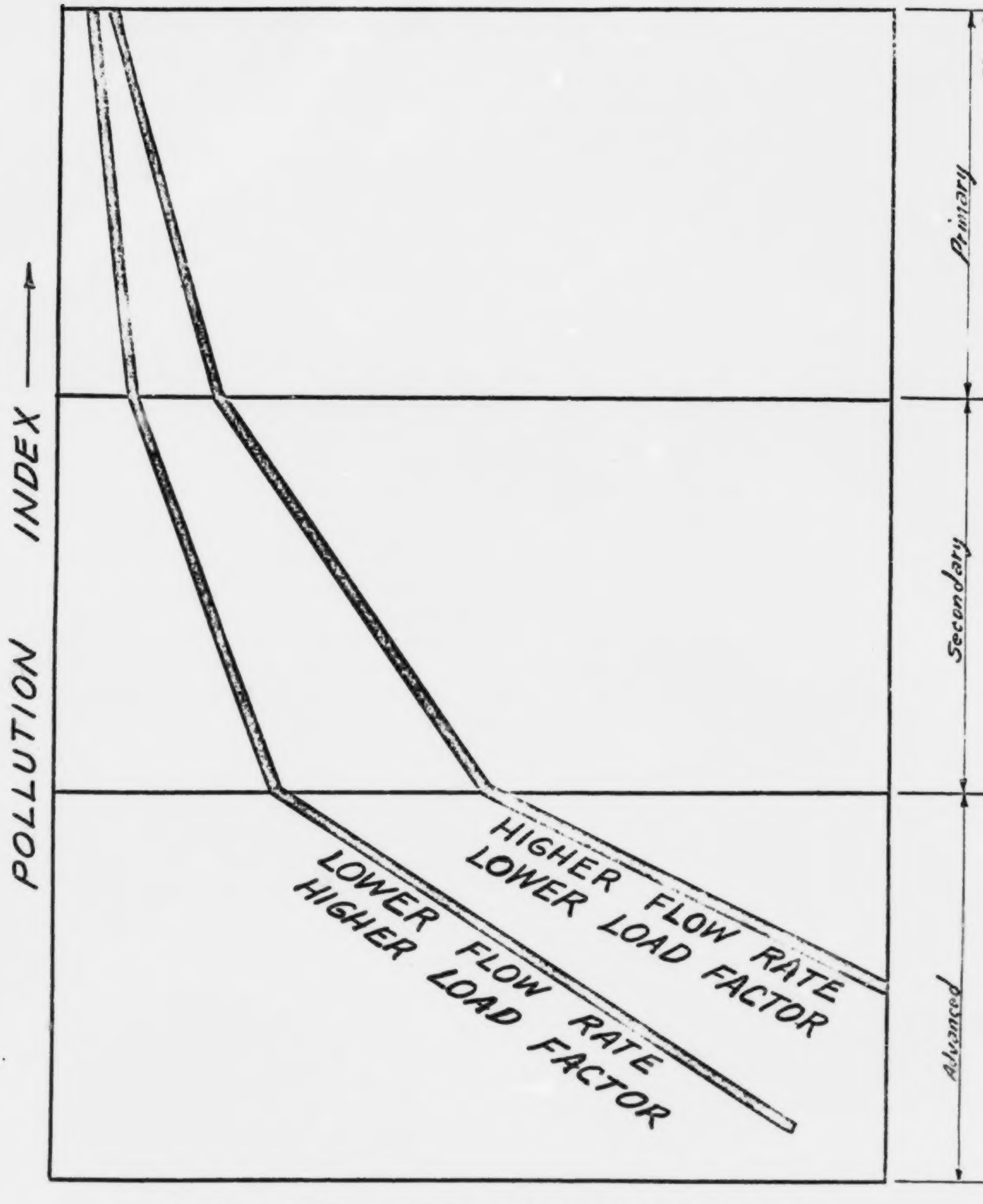
BAUER: Yes, we did make an analyses of that. If you did not have any storage and you were to design for two times dry weather flow, as I recall, the total combined treatment plant capacity (for the Chicago Sanitary District) would be 3600 mgd. If we were to--by regulating flows through these plants--reduce that required capacity to 1.25 times dry weather flow, then the saving would be something like 1350 mgd. Now the cost of this incremental plant capacity that would not have to be constructed, assuming advanced forms of treatment would be required, would be say $400,000 times 1350 = $540 million. If that is the ballpark figure that we are in, and there is a savings of something on the order of $500 million in treatment plant capacity, because of the regulation afforded by storage, that becomes a very significant cost factor. It was not really emphasized in the work that has been done so far; I think partly for the reason that there was no pilot plants to demonstrate this effective reduction. On the basis of the analyses made here, I am assuming that we do not have the kind of effect that Mr. Leary mentioned; but, I think that kind of effect has to be evaluated before you could say for sure that you could save that much in treatment plant capacity. At any rate, it is a large number.

BACON: I think there is something else that should be added here; it is not just a matter of cost. In the Chicago area where they have gone quite far in a rapid sand filtration of a rather crude type and with micro-strainers, it has been demonstrated that we can no longer tolerate these wild fluctuations in the secondary effluent ranging from 15 ppm BOD and 25 ppm of suspended solids to twice those figures and still come off with a consistent tertiary effluent.

BAUER: I think we found that if we did get into micro-strainers which could be an extremely economical type of tertiary treatment, they just will not take an inconsistent or double loading from a secondary treatment. So, in a sense the high standards of 4 ppm of BOD and 5 ppm of suspended solids cannot be achieved without storage. You are going to have to go into tertiary facilities with something more uniform and, therefore, it is not only a matter of cost figures, but a matter of consistency of the load application to the tertiary facility. I do not see that there is any doubt in the world in the Chicago situation that this fact has been demonstrated. Consequently, whether you are going to use storage to get a more uniform result in flow to the tertiary or whether you are going to go back to the primary and secondary and use such things as chemical treatment to try to come out of the secondary with a more consistent effluent remains to be seen, but, it is more than just a matter of cost.

I neglected to mention that in the Chicago tunnel plan there are also some surface storage facilities which are large dyked areas for the storage storm water. These have a much lower unit cost than I have calculated here and I think if it is possible in any area to use an uncovered open earthen vessel for the temporary storage that it is so economical as to make the storage extremely attractive. I assumed in the cost analyses in this report that it would not be possible; it would require either a concrete box with a lid on it or underground storage. But is is true that the regulation of the flows has a great deal of benefit.

Fig. 1


POLLUTION INDEX
HIGHER FLOW RATE
LOWER LOAD FACTOR
LOWER FLOW RATE
HIGHER LOAD FACTOR
Primary
Secondary
Advanced
DOLLARS PER MILLION GALLONS

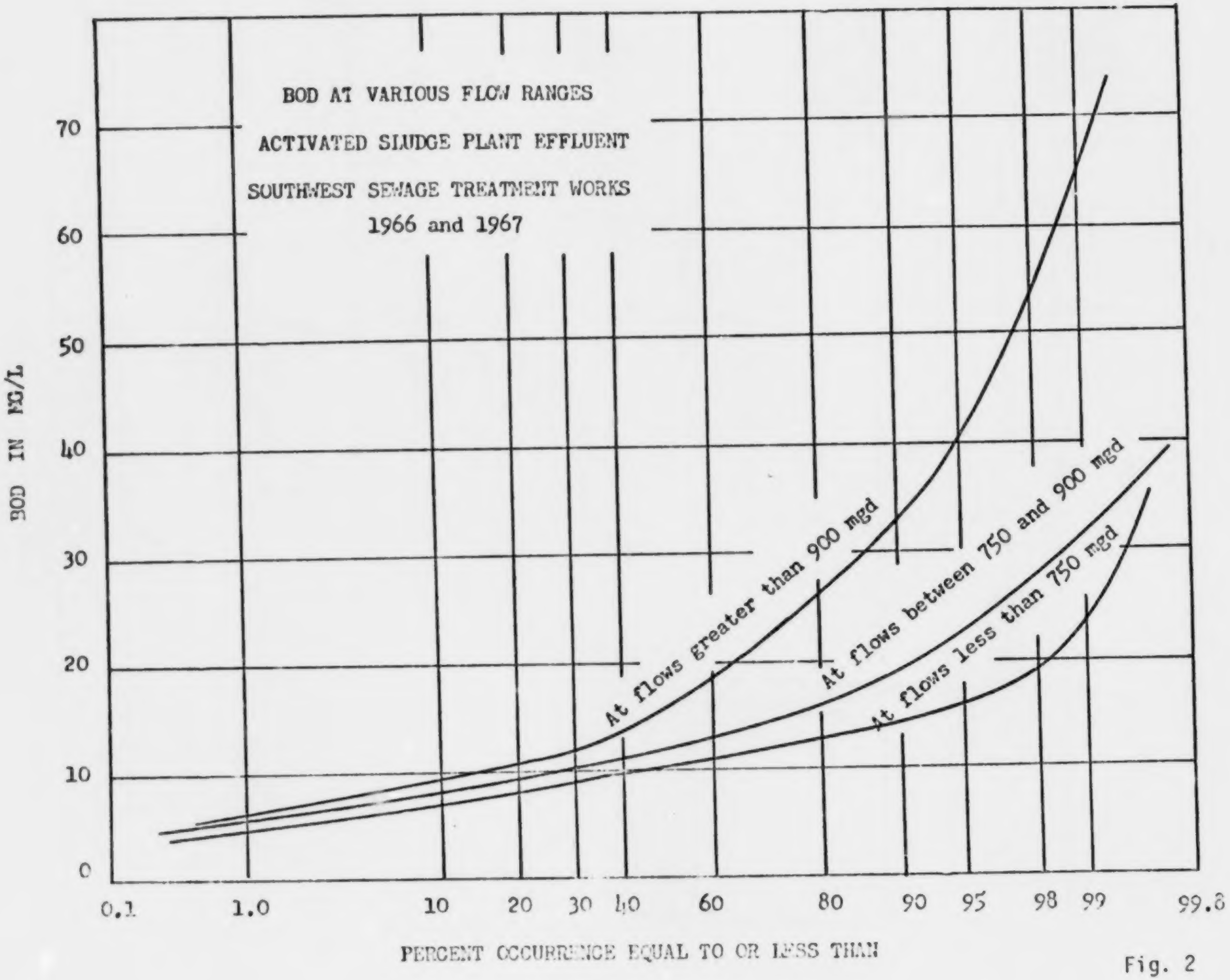
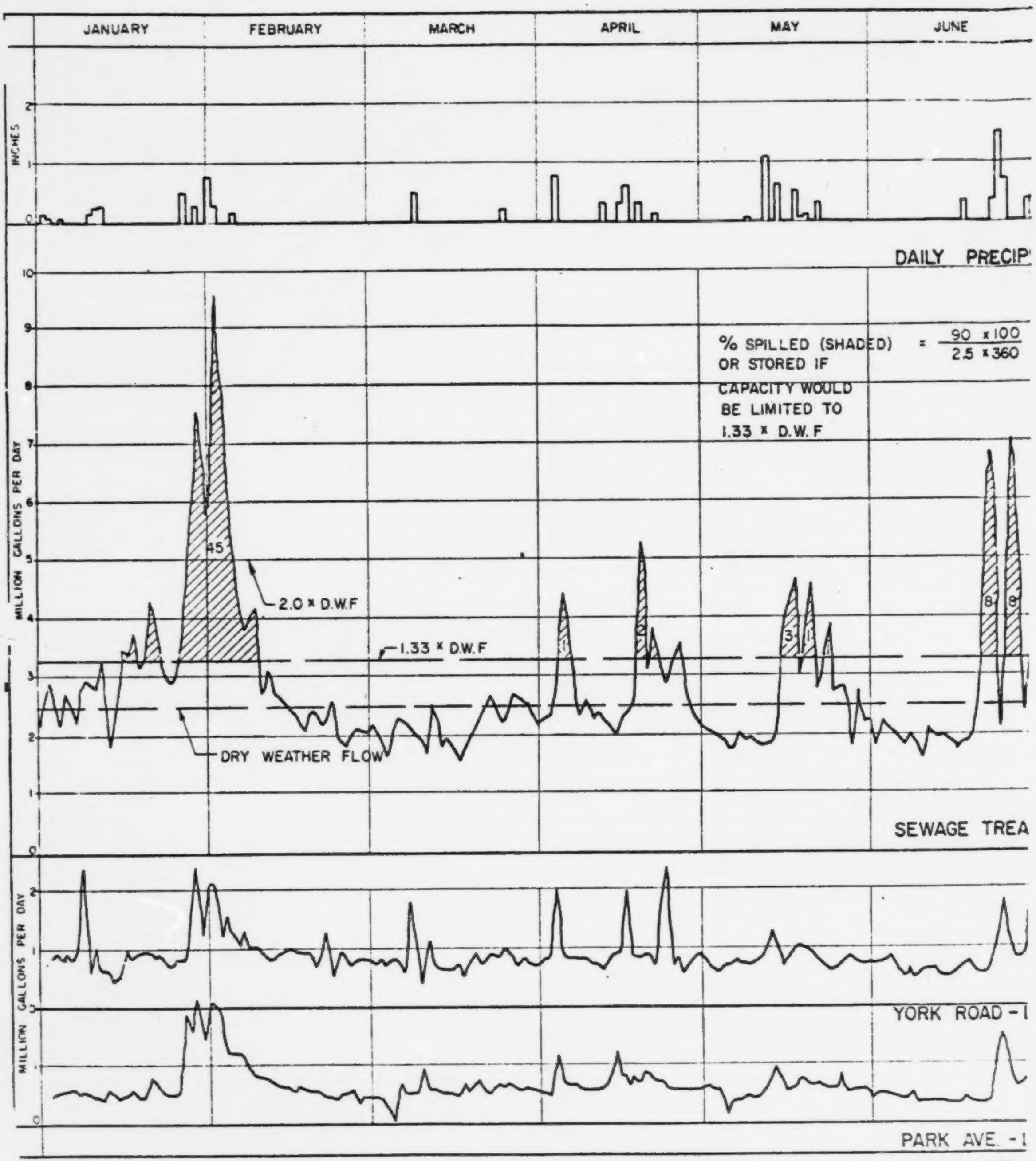

BOD AT VARIOUS FLOW RANGES
ACTIVATED SLUDGE PLANT EFFLUENT
SOUTHWEST SEWAGE TREATMENT WORKS
1966 and 1967
BOD IN MG/L
0
10
20
30
40
50
60
70
0.1
1.0
10
20
30
40
60
80
90
95
98
99
99.8
PERCENT OCCURRENCE EQUAL TO OR LESS THAN
At flows greater than 900 mgd
At flows between 750 and 900 mgd
At flows less than 750 mgd

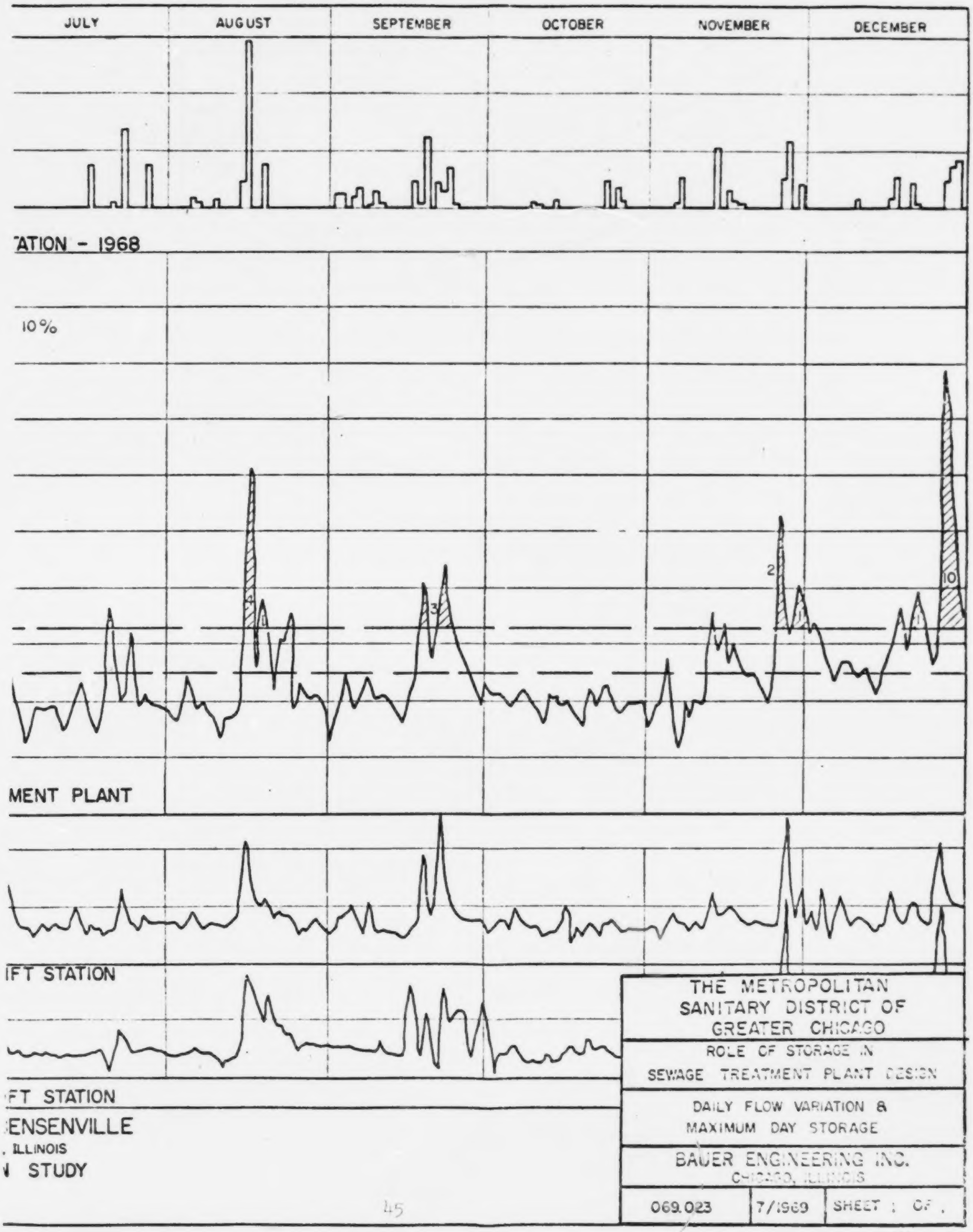
Fig. 2



% SPILLED (SHADED) OR STORED IF CAPACITY WOULD BE LIMITED TO 1.33 × D.W.F $= \frac{90 \times 100}{2.5 \times 360}$

TIME

VILLAGE OF
BENSENVILL
INFILTRATIC



THE METROPOLITAN
SANITARY DISTRICT OF
GREATER CHICAGO

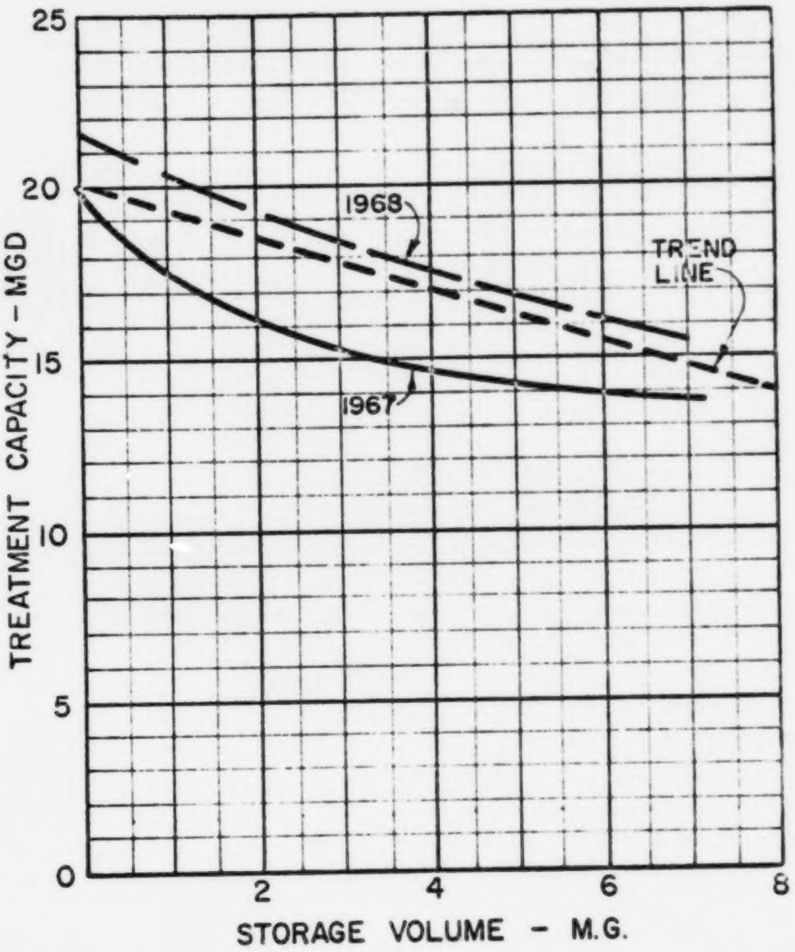
ROLE OF STORAGE IN
SEWAGE TREATMENT PLANT DESIGN

DAILY FLOW VARIATION &
MAXIMUM DAY STORAGE

BAUER ENGINEERING INC.
CHICAGO, ILLINOIS

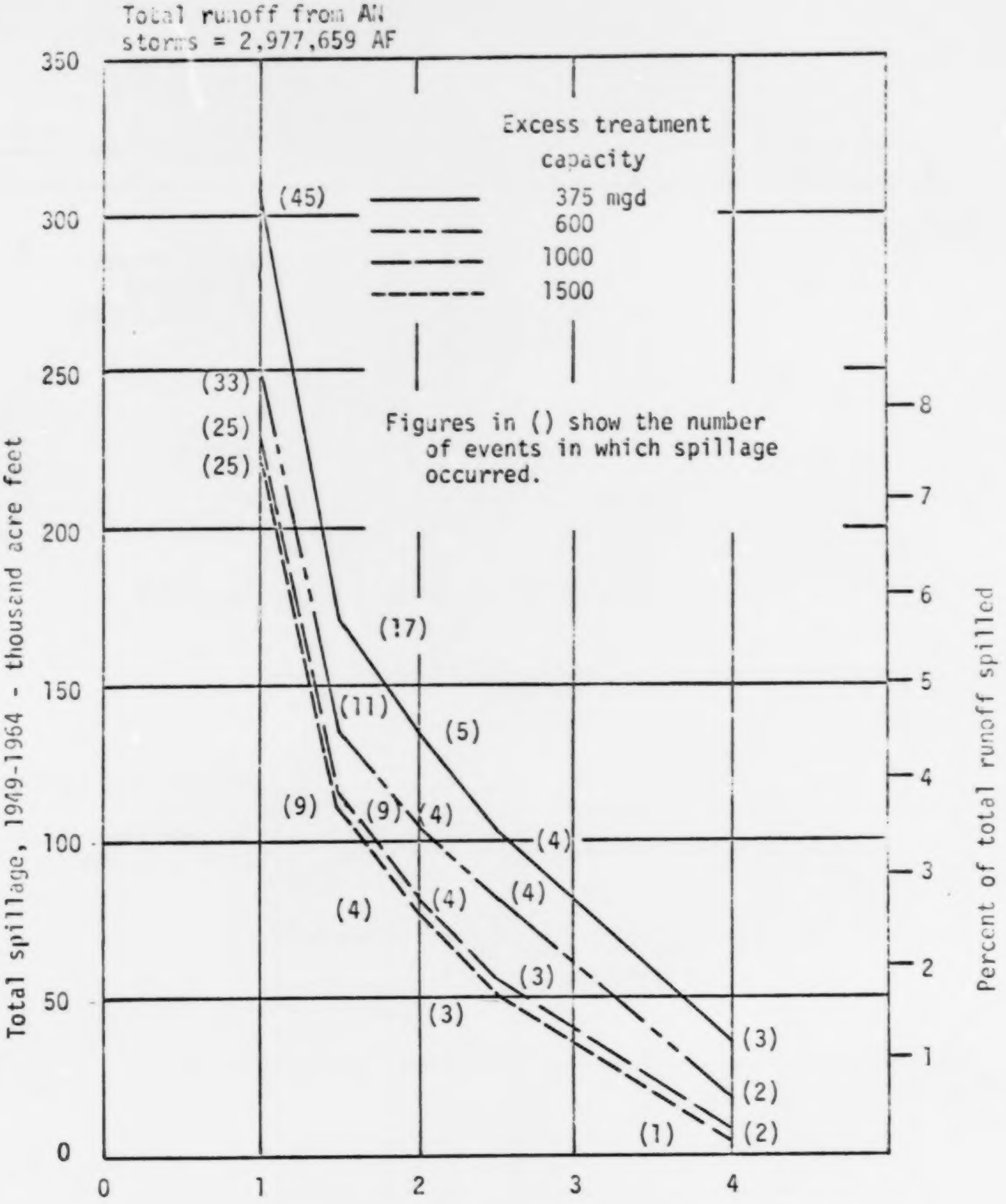
069.023 | 7/1969 | SHEET : OF .

Fig. 3



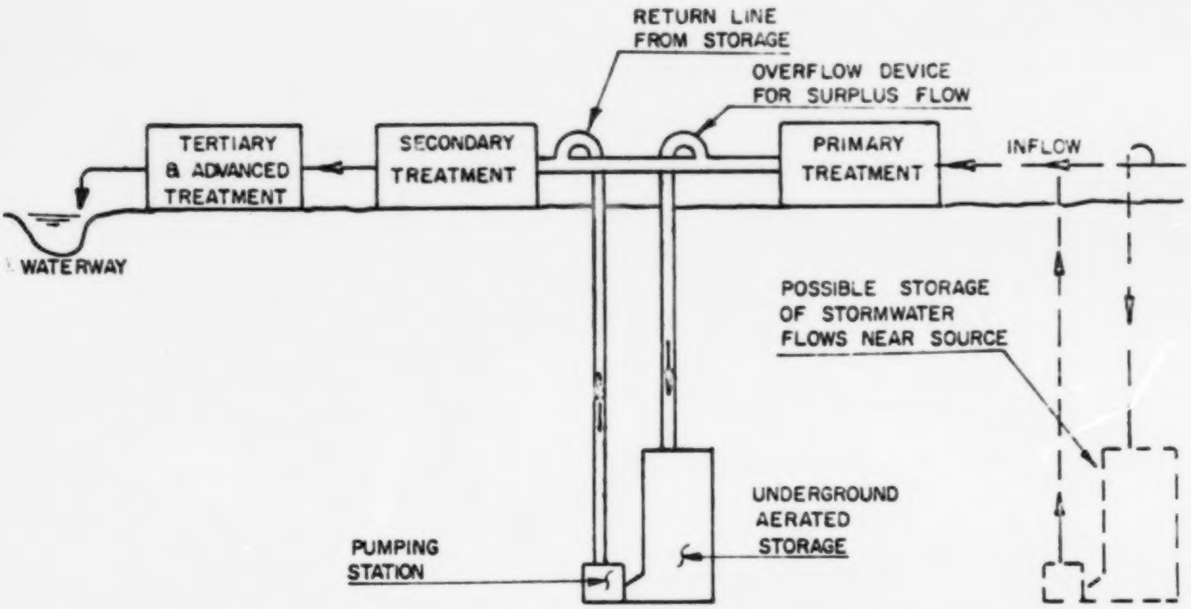
NOTE: BASED ON ANALYSIS
OF SANITARY DISTRICT
OF BLOOM TOWNSHIP
OPERATING RECORDS.

Fig. 4

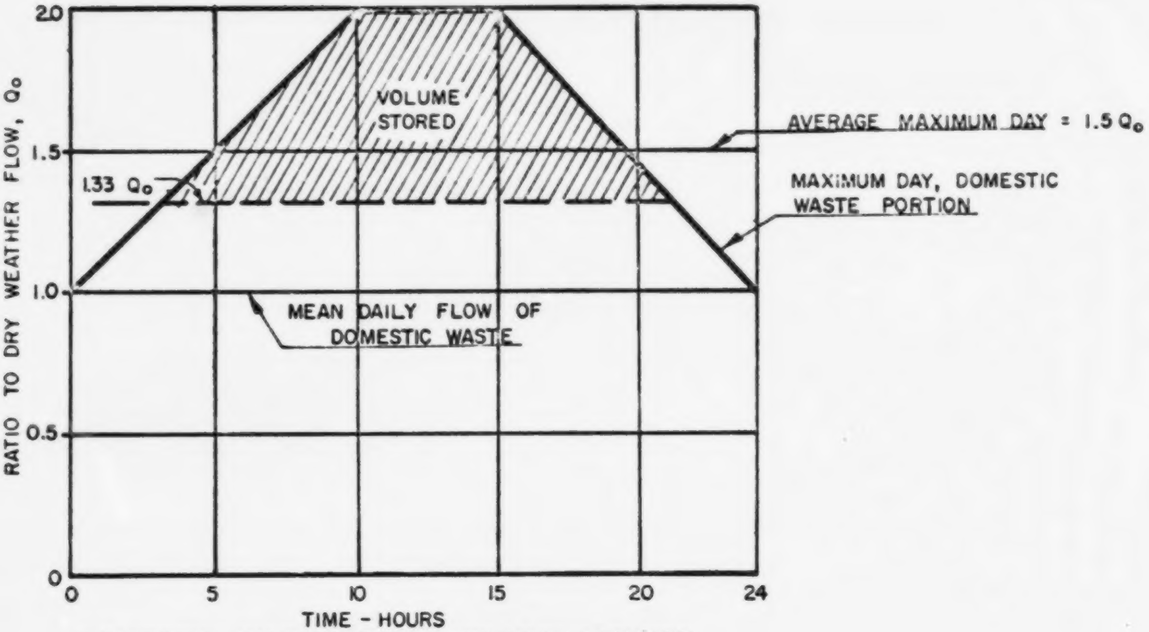


Total Spillage VS Storage Capacity

Fig. 5


RETURN LINE FROM STORAGE
OVERFLOW DEVICE FOR SURPLUS FLOW
TERTIARY & ADVANCED TREATMENT
SECONDARY TREATMENT
PRIMARY TREATMENT
INFLOW
WATERWAY
POSSIBLE STORAGE OF STORMWATER FLOWS NEAR SOURCE
UNDERGROUND AERATED STORAGE
PUMPING STATION

SIMPLIFIED SYSTEM FLOW DIAGRAM



DOMESTIC WASTEWATER VOLUME STORED DURING MAXIMUM DAY

Fig. 6

Section 4

THE IMPACT OF THE DEEP TUNNEL PLAN ON WATER RESOURCES IN THE CHICAGO AREA

by

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Chief, Engineering & Environmental Science
National Water Commission
Arlington, Virginia 22203

THE WATER CONSERVATION ASPECTS OF THE DEEP TUNNEL PLAN FOR THE CHICAGO AREA

SUMMARY

The Harza-Bauer proposal of a Deep Tunnel Project for the Metropolitan Sanitary District of Greater Chicago is designed to provide temporary storage for storm water and its accompanying pollution load. Tunnels and storage areas would be excavated in solid rock at elevations varying from 250 to 800 ft. below ground level. They would hold storm runoff which now floods basements and viaducts and pollutes streams in the area. On cessation of the storm, the stored water would be pumped to the surface and then to the District's treatment plant. After treatment, it would be returned to the rivers and streams of the area.

The attached paper describes the impact of the Deep Tunnel Project on two aspects of the water resources of Northeast Illinois - surface water and ground water. It shows that the benefits to conservation of water could be as significant as those originally expected for flood and pollution control.

For surface water, it is estimated that the Deep Tunnel Project would, in effect, ultimately make available an additional 515 cfs (332 mgd) for use in the Northeast Illinois, because of better regulation and complete treatment of storm water overflows. This compares with about 1,700 cfs of present pumpage from Lake Michigan for domestic and industrial use. The value of this water, when fully used, is estimated to range from $3.6 million to $6.0 million annually for each 100 cfs (65 mgd), depending on alternatives. This would justify a capital investment, if staged to meet uses, of $18 million to $86 million, depending also on interest rates to be used, for each 100 cfs.

For ground water, the paper describes elaborate measures planned to protect the aquifers, presently sources of about 130 mgd of the metropolitan area supply, from pollution by the Project. It demonstrates how the Project could serve as a management vehicle to reverse the trend of ground water "mining" in the metropolitan area.

My paper is on the subject of the water conserving aspects of the Deep Tunnel Project for Chicago, made possible by the complete treatment and controlled releases of storm water overflows. The National Water commission, of course, has a very great interest in conservation of water--in fact, one of our principal studies is on methods of conserving water.

However, while I, as a member of the Commission's staff, have great interest in the Deep Tunnel Project from that viewpoint, I must make it clear that this paper stems, not from any studies I have been associated

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with while with the Commission--rather it derives from studies made under my supervision while I was with the Harza Engineering Company. Most of the information was contained in a 1969 Harza-Bauer report[1]. The views expressed hereafter are my own, and not those of the Commission.

THE DEEP TUNNEL PROJECT

The details of alternative proposals for deep tunnels in the Chicago area have been described by other participants in this Institute. The concepts of water conservation that are presented in this paper are those which apply to the Harza-Bauer "Deep Tunnel Plan," as proposed to the Sanitary District[2]. Some of the concepts also might apply to other plans for deep tunnels--these will be touched on briefly later in this paper.

Although the details of the problem and of the Harza-Bauer plan are presented in other papers, a brief discussion is presented here for completeness.

THE PROBLEM

Nature has treated the Chicago area very poorly in providing for handling of storm water and the accompanying pollution load. The flat topography and the low gradients on most of the small streams have always caused difficulties in drainage. In its natural state, much of the Chicago area was a swamp.

The early sewer systems in the Chicago area were combined sewers, intended to handle both storm water and raw sewage. This practice has continued, for the most part, until the present. The present combined sewer system serves 300 square miles of heavily populated area.

In time of storm, the capacity of the sewer and treatment system is too small to handle both sewage and storm water. Therefore, during such periods relief is obtained by discharge of the mixture of storm water and raw domestic and industrial sewage to the Illinois Waterway system. The overflows from the combined sewer system enter the Waterway at some 400 locations, as shown on Figure 1.

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[1] "The Impact of the Deep Tunnel Plan on the Water Resources of Northeast Illinois," A Report by the Harza Engineering Company and Bauer Engineering, Inc., prepared for the Metropolitan Sanitary District of Greater Chicago, February 1969.

[2] Ibid.

When the overflows are too large for the Waterway system to accomodate, it is necessary, on rare occasions, to discharge this mixture of storm water and sewage to Lake Michigan. Such a discharge occurred on August 16, 1968, causing Chicago's beaches to be closed on one of the hottest weeks of the year. While such occasions have been rare (only four times in the last 25 years), they are detrimental to recreational activities of the area. Associated with this, on many occasions, has been flood damage along the waterway.

Locally, the increased runoff which has accompanied urbanization has overloaded the small sewer capacity before it can be relieved at their overflow points on the Waterway. In suburban areas, capacities of local streams which serve as outlets also are limited. Because of these limitations, relief occurs locally, both in the city and the suburbs, by temporary storage on streets, underpasses, and basements. Since this water is frequently polluted, it is a health hazard as well as a property damage hazard.

DESCRIPTION OF THE DEEP TUNNEL PROJECT

The general concept of the Deep Tunnel Project is relatively simple, as shown on Figure 2. It combines certain features of what has become known as the "City of Chicago underflow" concept with underground storage, treatment, and hydroelectric power generation. Basically, the Deep Tunnel Project involves:

a. Providing lower outlets for existing and proposed new main sewers and interceptors, which will increase sewer capacities by increasing their hydraulic gradients.

b. Intercepting, conveying, and storing combined sewer overflows that might otherwise overflow to the waterways.

c. Releasing the stored waters at a reduced rate, first to an advanced waste treatment plant and then to the waterway. This will virtually eliminate both pollution and flooding in the waterways due to storm water overflows.

Ultimately, the Deep Tunnel System is proposed to service the entire area of 300-square miles of combined sewers that are shown on Figure 1. The First Construction Zone, as originally proposed, would serve 62 square miles in the Lake Calumet area. Since the original scheduling, a second zone which would serve an area on the North Branch of the Chicago River has been planned for concurrent construction with the first zone. The proposed system for the entire service area, including the Calumet and North Side areas, is shown on Figure 3. The service area of the First Zone and a general layout of the project features are shown on Figure 4.

The Deep Tunnel System will start with the capture of storm water overflows from combined sewers at a point just upstream of the outfall to the waterway. These polluted overflows, instead of entering the waterway, will be dropped through vertical shafts into a network of smooth tunnels under the waterways, located at two principal levels, in the Niagaran and the Galena-Platteville dolomites. The tunnels, designed for flow under pressure (to utilize the large head available), will conduct the overflow water to a central, mined storage reservoir, some 830 feet below the land surface. The mined storage reservoir, made up of large, unlined chambers in the Galena dolomite, will consist of two sections, a settling chamber and the main storage reservior.

Water will first flow into a settling chamber, which will be large enough to contain runoff from small and medium storms, and retain much of the solids loads of large storm runoffs. The partially treated water will then flow into the main storage reservior, from where it will be pumped through reversible pump-turbines to a diked reservoir on the surface, using off-peak power. Storm water stored in the surface reservoir will further improve in quality due to sedimentation and oxidation, and will then be fully treated and released gradually to the waterways. In this process, the Deep Tunnel System will eliminate 99.5 percent of the pollution load presently reaching the waterways through storm overflows from the sewers.

Both the surface and lower reservoirs will serve the dual function of storm water retention and storage for hydroelectric power generation. These uses are compatible. Analyses based on 96 years of rainfall records indicate that hydroelectric generation would be curtailed less than 0.1 percent of its operating time due to storm water retention.

TYPES OF IMPACTS ON WATER RESOURCES

Although the Deep Tunnel Project was conceived initially as a flood control and pollution control project (with incidental power generation facilities), it has an impact on the water resources of the area of considerable importance. In the future, as the Chicago area becomes increasingly in need of more water, this impact could emerge as being fully as significant as the "primary" purposes of flood and pollution control.

There are two ways in which the Deep Tunnel Project would have an impact on the water resources of Northeast Illinois. The first is in conservation of surface water resources, the second is in management of ground water resources. Each will be dealt with separately in the following sections.

IMPACT ON THE SURFACE WATER SUPPLY

THE PRESENT SUPPLY

The surface water presently available to the Northeast Illinois area consists primarily of the supply from Lake Michigan. Prior to 1967, there was no limitation on the amount of domestic and industrial pumpage from Lake Michigan by the City of Chicago and a privileged few of its suburbs. However, the 1967 decree of the U.S. Supreme

Court has made the surface water supply much more critical, by introducing new limitations. While the amount of water that could be pumped from the Lake would be unlimited if it were possible to return the used water to the Lake, such return is considered unacceptable under present conditions because of potential pollution of the Lake. Under present conditions, the only acceptable procedure is to direct the water, after it is used, down the Illinois Waterway.

The amount of diversion from the Lake to the Waterway is now limited by the 1967 decree to a total of 3200 cfs (2,060 mgd). The decree defines three components that must be counted in such diversion. These components are as follows:

a. Domestic pumpage (including water supplied to commercial and industrial establishments but excluding well pumpage), the sewage effluent derived from which is not returned to Lake Michigan. This component has been estimated to average 1,734 cfs (1,110 mgd) during the 1950-1964 period.[3]

b. Storm runoff which, without the interception by the canal system of the Metropolitan Sanitary District, would have entered Lake Michigan from the natural drainage of the Lake Michigan watershed. This component has been estimated to average 550 cfs (355 mgd) during the 1950-1959 period.[4] This was prior to construction of the O'Brien Lock, when the area intercepted in this manner was about 450 square miles. The Sanitary District feels that the Special Masters' estimates are high--that the correct figure is more in the order of 400 cfs. The District feels that the 550 cfs is more applicable to current conditions, with O'Brien Lock, where the area of normal Lake Michigan drainage that is intercepted is about 740 square miles.

c. Direct diversion from the Lake into the canal system of the Sanitary District, estimated to average about 945 cfs for the 1950-1964 period.[5] (Since the total of the three components has historically been 3,229 cfs, it can be presumed that this component would have to be reduced by 29 cfs, to 916 cfs, to comply with the decree.)

The total amount of water to be diverted from Lake Michigan (3,200 cfs) presently serves the purposes of (1) municipal and industrial water supply, (2) maintaining sanitary conditions in the Illinois Waterway, and (3) navigation. The 1967 decree of the Supreme Court specifies that the State of Illinois may apportion the 3,200 cfs among these uses as it sees fit, subject only to any regulations imposed by Congress in the interest of navigation or pollution control. Thus, the 1,734 cfs of

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[3] Report of Albert B. Maris, Special Master, to Supreme Court of the United States, Wisconsin et al, vs Illinois et al, Dec. 8, 1966, p. 87.
[4] Ibid., p. 87.
[5] Ibid., p. 87.

"domestic pumpage" can be increased if the needs of navigation and pollution control are adequately cared for with their reduced supply.

In addition to surface water diverted from Lake Michigan, there is an undeveloped source of water in streams naturally draining away from Lake Michigan. This supply is quite erratic in its occurrence, and would require substantial storage facilities for regulation. Major streams which might be utilized in this manner include the Des Plaines, the Fox, and the Kankakee. Reliable cost estimates are not available for the storage and transmission systems that would be required, but they undoubtedly would be very expensive.

Another method of utilizing the above rivers, proposed by the Lake States which opposed Illinois in the suit before the U.S. Supreme Court, would be to divert water into Lake Michigan from the rivers to compensate for water diverted from Lake Michigan. The cost estimates by the Lake States are believed by Illinois to be quite low and the feasibility of making compensating diversions from the three basins, of the magnitude suggested, are open to serious questions.

THE NEED FOR WATER MANAGEMENT

The 1967 decree of the Supreme Court provides that application for modification of its terms can be made only if the State of Illinois demonstrates (a) that the ground and surface water resources of the region are not adequate to meet the needs, and (b) that all feasible means that are reasonably available have been employed to conserve and manage the water resources of the area.

The terms of the 1967 Supreme Court decree are very positive in stating what Illinois must do if it ever wishes to obtain more than 3,200 cfs from Lake Michigan. Mr. W. C. Ackermann has been very active in pointing this out. He has stated:

> It is perfectly clear, however, that the State of Illinois has the duty so to manage its water resources and regulate the use of the water now available to it as to conserve this essential commodity to the utmost practicable extent for the use of its people. This surely means that the State must definitely undertake the task of managing its water resources, at least in its Northeastern Metropolitan Region, on a broad regional basis in the most modern scientific manner and that all feasible methods for developing the supply and conserving the use of domestic water which are reasonably available to it should be employed, before the State receives authority to divert more than the present 3,200 cfs from Lake Michigan.[6]

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[6] "Implications of the Maris Report," W. C. Ackermann, Chief Illinois State Water Survey, A talk prepared for the Great Lakes Water Resources Conference in Toronto, Canada, June 25, 1968.

CONSERVATION OF STORM RUNOFF FROM DES PLAINES WATERSHED

In addition to the stormwater overflows in the Lake Michigan natural watershed, significant overflows that occur in Des Plaines River watersheds will be captured by the Deep Tunnel Project and treated. These overflows are from an estimated 60 square miles of the 300 square mile combined sewer area. These overflows are estimated to be the same as from the Lake Michigan watershed, on a cfs-per-square-mile basis, and thus would be about an average of 50 cfs (32 mgd).

Since the origin of this water is from outside the Lake Michigan watershed, it is not charged against Illinois' diversions from that watershed. In fact, since it would ultimately be discharged into Lake Michigan, or accomplish the equivalent thereof, it would be an import into the basin, to serve as a credit against diverted water.

The Sanitary District has recently adopted resolutions which eventually could lead to providing flood control storage on a number of small streams, outside the combined serviced area, which enter the Waterway. These streams, which drain approximately 1,260 square miles, would be controlled in a manner that would require releases to be made in a matter of a relatively few hours, or days at most, after a storm. Thus their releases could not be used directly as a supply of water.

It seems possible, however, that as the Northeast Illinois area becomes more restricted by the limits of the decree in relation to growing water needs, some coordinated operation of the Deep Tunnel system and such flood control reservoirs could be accomplished. Flood releases from these reservoirs might be made directly to the Deep Tunnel system for all except the major storms, in effect creating additional imports to the Lake Michigan watershed. Such coordination of operation probably could be made feasible, without any loss of dependable power capacity, because it could be done at a time in the future when the power load curve is such that the number of kilowatt-hours of energy (equivalent to the water storage volume usable for power) would be significantly less than initially.

CONSERVATION OF PRESENT DIRECT DIVERSIONS TO THE WATERWAY

The water diverted directly to the waterway system (estimated to average 945 cfs, but assumed to be decreased to 916 cfs to comply with the decree) is used for navigation and maintenance of sanitary conditions. Although the Corps of Engineers has not stated what it considers to be the minimum amount necessary for navigation, it has stated that the total present diversions would be adequate to meet future navigation requirements, with the improvements it contemplates for the future.

The Deep Tunnel Project will give complete treatment to storm water overflows, which have contributed heavily to unsanitary conditions in the waterway. The Sanitary District also has programs aimed at eliminating the adverse effects of the other two main sources of waterway pollution (sewage plant effluent and industrial wastes). The present sewage plant effluents are expected to be given advanced waste treatment processes. The industrial waste pollution is being eliminated by measures taken by industry, under the legal requirements of the recently adopted waterway standards being enforced by the District.

After these steps are taken, there will no longer be a need for diversions to maintain sanitary conditions. There will remain a need for direct diversions to serve navigation, since water will be needed for lockages at the mouth of the Chicago River and at the O'Brien Lock on the Calumet River. There will also be leakage through these locks. The combined amounts of water needed for such lockage and leakage at the mouth of the Chicago River and at the O'Brien Lock has been estimated as being 130 cfs[7]. The Sanitary District has estimated the requirements for lockage and leakage at Wilmette to be 20 cfs. Thus, the total lockage and leakage requirements would be about 150 cfs (100 mgd).

It probably would be possible to conserve most of the lockage and leakage. However, as a minimum, there would be in the order of 765 cfs (the 916 cfs of direct diversions diverted minus 150 cfs) which could be added to the region's usable water resources if all three sources of pollution were eliminated. The Deep Tunnel Project cannot claim credit for all of these savings, since other measures are also necessary. However, it could, on the basis of percentage of pollution eliminated, claim credit for conservation of about one-third, or 255 cfs (165 mgd).

EFFECTS ON NAVIGATION DOWNSTREAM

The 210 cfs of storm water runoff in the Lake Michigan watershed, plus the 50 cfs originating outside the watershed, if converted to a usable water supply, would eventually return to the Illinois Waterway and would be uniform in flow, rather than occurring erratically in storm periods, as at present. This would benefit navigation significantly. The 765 cfs of direct diversions that is saved, if used as municipal and industrial water supply, also would return to the waterways. Thus the total average water available at Lockport and downstream locations would be same as at present, but would be better regulated and vastly improved in quality.

SUMMARY OF EFFECT ON SURFACE WATER SUPPLY

On the basis of the above estimates, the Deep Tunnel Project would make available the following additional quantities of water for municipal and industrial use:

From the storm water originating within the Lake Michigan watershed	210 cfs	135 mgd

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[7] Maris Report, p. 86.

From storm water originating outside the Lake Michigan watershed	50 cfs	32 mgd
Savings in present direct diversions	255 cfs	165 mgd
TOTAL	515 cfs	332 mgd

This increase in usable water supply from the existing water resources of Northeast Illinois is more than one-half the 930 cfs of additional water needed to serve the growth in population in the Metropolitan area by the year 2000, as estimated in the report by the Special Master[8].

STAGING OF USE OF ADDITIONAL WATER

The water conserving potential of the Deep Tunnel Project is ideally suited to staging of use of the water, as the needs of the Chicago Metropolitan area grow.

The plan outlined herein is based on the Harza-Bauer plan of 1968. Since that time, it is understood that agreement has been reached to proceed with an "underflow" concept for an area of the North Shore channel, from approximately the junction of the North Branch of Chicago River to the Wilmette inlet. While this will have less storage (about 1.25 inches as compared to the 2.2 inches provided in the Deep Tunnel Project), it will capture a significant percentage of the overflows. Since the captured overflows will be given additional treatment, their release to the waterway will contribute to the cleanup program.

The first stage of conservation through the Deep Tunnel system might be the substitution of the storm water releases that are treated in the North Shore channel phase for an equal amount of direct diversions, presumably on the basis that equivalent sanitary conditions would obtain. The writer does not have specific knowledge of the area to be served by the North Shore channel phase, but it could provide a saving of 10 to 15 cfs (6 to 10 mgd), sufficient for a population of about 70,000 to 100,000 (at a usage rate of 100 gallons per capita per day, which is higher than the present usage in most Chicago suburban areas).

The next step could be upon the completion of the initial storage for the Deep Tunnel Project, under the mined storage concept conceived in the Harza-Bauer Plan. This could either be in the Calumet area or in the McCook area (see Figure 3), either of which seems to be feasible extensions of the underflow phase for the North Shore channel area. At that time the additional water salvaged, approximately 40 cfs (25 mgd) if the First Zone of Figure 4 is developed; could be made available immediately by releasing it directly to the waterway as a replacement for an equal amount of direct diversion. Since it would be fully treated, the 40 cfs that would be saved could be used for municipal and industrial purposes.

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[8] Maris Report, p. 102.

Ultimately, when the Deep Tunnel Project is completed, the 260 cfs (168 mgd) of storm water runoff that would be salvaged could be put to use in the same way. This amount would be sufficient to support an additional 1,700,000 persons (at 100 gallons per capita per day).

The next step after use of the 260 cfs would be to utilize all of the savings in direct diversion that can be used without installing additional transmission facilities. The 260 cfs of captured storm water will not be released uniformly to the waterway throughout the year--our studies indicate this would vary from zero to a maximum of 562 cfs. The direct diversions would be increased or decreased as necessary to compensate for the changes in rate of release of treated storm flows. Any direct diversions that may be saved as a result of complete treatment of all sources of pollution (storm water overflows, waste treatment plant effluents, and industrial wastes) would be staged after the preceding stages are utilized.

Additional future steps could involve conveyance of all the captured storm water back to the lake for regulation to uniform flow throughout the year, or capture of additional storm water from reservoir releases in areas outside the combined sewer area. Thus the Deep Tunnel Project fits in ideally to a staged development of providing additional water supply, as the population of the area grows.

VALUE OF WATER CONSERVED

The value of the resource that would be conserved is dependent on the alternative cost of water supply that might be available. This has not been determined reliably for the Chicago area, but several yardsticks might be used as the basis for an appraisal. These are:

a. The average cost of water delivered to all communities in the U.S. (including transmission but not distribution costs), estimated to be 12 cents per 1,000 gallons.

b. The anticipated cost of the necessary steps in advanced waste treatment (over the cost of secondary treatment) to provide water suitable for reuse for municipal and industrial purposes, which is estimated to be about 25 cents per 1,000 gallons (excluding transmission and distribution).[9]

c. The cost of providing water to suburban areas (including treatment), by pumping from ground water, estimated to be 15 to 25 cents per 1,000 gallons.

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[9] From information in draft of report on this subject by National Water Commission.

In addition to the above, the Lake States estimated the costs for water diverted to Lake Michigan from the Fox, Des Plaines, and Kankakee Rivers, to be 2-1/2 to 6 cents per 1,000 gallons, not including treatment. As indicated previously, these estimates are believed to be quite low and, therefore, have not been used in the subsequent analysis.

The justifiable capital expenditures for measures to provide the additional water (assuming it were immediately useful upon completion of the facilities) would also depend on the terms of amortization of the facilities (primarily on the interest rate assumed) and the allocation of operating costs to the water conservation purpose. For purposes of illustration, the justifiable capital costs are presented in the following table on the basis of annual charges (debt service plus operating costs) of 7, 10, and 12 percent.

TABLE #

Value of Conserving each 100 cfs (65 mgd)

Source	Alternative Costs (Cents per 1,000 gal.)	Annual Value (millions of dollars)	Justified Capital Expenditure for various annual charges (in millions of dollars)		
			7%	10%	12%
Average in U.S.	12	2.1	30	21	18
From Advanced Treatment	25	6.0	86	60	50
Present Cost to Suburbs	15-25	3.6-6.0	51-86	36-60	20-50

It must be recognized that the above values are attainable only when there is a definite market use for the water, at the prices indicated. Such uses will build up only gradually, over a period of years--hence the values are shown for 100 cfs increments, to avoid the inference that the complete capital expenditure to provide a total saving of 515 cfs (332 mgd) is immediately justified. It will be necessary to make a reliable appraisal of the buildup in uses and the resulting cash flow, to develop a dependable basis for the justifiable staging of capital expenditures.

IMPACT ON THE GROUND WATER SUPPLY

EXISTING RESOURCES AND USE

The principal aquifers supplying the metropolitan area are, in order of depth, the sand and gravel deposits of the Glacial Drift, the Silurian dolomites limestone, the Cambrian-Ordovician sandstones and dolomites, and the Mt. Simon sandstone. A description of these systems is given on Figure 5.

In 1967, total ground water pumpage in an eight-county area of Northeast

Illinois was 243.7[10]. Of these, 29.5 mgd came from sand and gravel wells, 84.3 mgd from shallow dolomite (Silurian) wells and 129.9 mgd from deep sandstone wells. It is estimated that of the 129.9 mgd pumped from deep wells, 74 mgd came from the Cambrian-Ordovician aquifer and 55.9 mgd from the Silurian and Mt. Simon aquifers, through wells which are also open to these two aquifers.

Over 175 municipalities obtained their supply from ground water in 1967, using a total of 148.5 mgd. The remainder of ground water usage was by industries, 61.8 mgd, and by irrigation and domestic users, 33.4 mgd.

The Silurian system has been estimated by the Northeast Illinois Planning Commission to have a supply considerably in excess of present pumping requirements. However, this system is quite erratic in the occurrence of ground water at specific locations, especially within the metropolitan area, so that there is considerable risk of dry holes when wells are drilled. For this reason, the Silurian aquifer is only partially used.

Contrasted with this, the Cambrian-Ordovician aquifer is highly dependable as a source and also has a lower hardness. A village or industry can be assured of a good yield when it makes the investment necessary to drill a well. This aquifer system, therefore, is the most widely used and pumpage from this system has increased from about 25 mgd in 1940 to 74 mgd in 1967. Major centers of heavy pumping are at Des Plaines, Elmhurst and Summit. Because of this heavy usage, the withdrawals of ground water from the Cambrian-Ordovician system have exceeded the supply available through natural recharge, by some 60 percent. As a result, water levels have declined from artesian flow in 1864 to depths of 650 ft. in 1966. The rate of decline of the ground water level, averaging about 13 feet per year over the area, is indicative of the difficulties which the area will encounter if the "mining" of ground water continues. A water level map for the area is shown on Figure 6.

PROTECTION OF THE GROUND WATER RESOURCE

The tunnels and mined storage area of the Deep Tunnel System would be excavated in the two dolomite rock formations underlying the area (Niagaran and Galena-Platteville), at approximately 250 and 800 foot depths below the ground surface. The tunnels would be in these two separated rock units, which are part of two completely separate ground water systems, while the mined storage area would be only in the lower aquifer. The proposed Deep Tunnel System includes elements which have been designed to protect these aquifers from any deleterious effects of the storm water runoff that would be conveyed in the tunnels and stored in the mined area.

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[10] Personal communication, R.T. Sasman, Illinois State Water Survey, to Harza Engineering Company.

The principle on which the protection is based is extremely simple--even though the implementation of that principle is somewhat complicated. The principle is simply that water will not flow "uphill," that is, against a positive pressure. The principle is demonstrated in Figure 7. It will be implemented by assuring that the water pressure in the ground water surrounding the tunnels and mined area will always be greater than the pressure inside the tunnels and mined area. If this is the case, any flow of water that occurs must be inward, toward the tunnels, not outward. Under such conditions, pollution of the aquifer cannot occur.

The protective system that has been devised has been based on extensive ground water investigations, including an electric analog computer and field drilling, seismic surveys, well logging and pump tests, all of which were obtained at a cost of about $2,000,000.

The upper level tunnel system in the Harza-Bauer plan would be below the water level in the upper aquifer system. The tunnels would not be allowed to become pressurized above the outside ground water pressure. The upper aquifer is used only slightly, and, as a result, the supply of ground water available from infiltration of precipitation on the land surface, i.e., natural recharge, is much larger than present pumpage from this aquifer. It is also anticipated that future pumpage from this aquifer would not exceed the natural recharge. Therefore, the ground water pressure in the upper aquifer system would remain greater than the pressure in the tunnels, and pollution of the aquifer cannot occur, as shown on Figure 7(A).

Protection of the lower, completely separate, aquifer would require slightly different measures. The ground water in this aquifer is presently above the elevation of the lower tunnels and the mined storage area; however, as indicated previously, because of heavy usage, the ground water level is gradually falling, at an average rate of about 13 feet per year throughout the area. To adequately protect the lower aquifer, this trend must be reversed so that the piezometric level of the ground water will always be above the level in the tunnels and the mined storage area. The principle of protection for the lower aquifer is demonstrated on Figures 7(B) and 7(C).

Two measures are possible to maintain the pressure in the ground water above that in the tunnel and mined area. These are (1) artificial recharge of the aquifer, and (2) control of ground water pumping.

A recharge system is entirely feasible, would use relatively small quantities of surface water for recharge, and would not necessitate the exchange of a surface water supply for a ground water supply with present users. Most of the recharge water would move toward existing wells, thus augmenting the available supply. The validity of the recharge approach has been demonstrated by studies which included a $487,000 ground water drilling and testing program, construction of an electric analog computer, and office evaluation of collected data and analog results. Additional subsurface information, obtained through the geologic investigations of the area, included a $437,000 diamond core drilling program, and a $1,006,000 seismic exploration and geophysical well logging program. The results of all these field investigations were incorporated into the ground water studies.

The second method of protection, which may ultimately prove to be highly practical, would be to manage ground water pumping in the area in such a manner that ground water levels will be maintained at a high enough level to assure that the water level outside the storage area is higher than inside. This would require substitution of a surface water source of supply to a portion of the users now pumping ground water. As explained previously, it is believed that the Deep Tunnel Project would ultimately make a large amount of additional surface water available, so that this method would be highly promising and entirely practicable, from an engineering viewpoint. A number of complex administrative arrangements would be necessary to implement this approach. In effect, this would require overall, centralized management of all of the water resources of the Northeast Illinois area. Such management arrangements would be complicated, but would produce significant overall benefits.

COORDINATION WITH STATE OFFICIALS

Throughout the course of the studies underlying the Harza-Bauer Plan, Mr. William C. Ackermann, Chief of the Illinois State Water Survey, and Mr. Clarence W. Klassen, Chief Sanitary Engineer of the State Department of Public Health and Technical Secretary of the State Sanitary Water Board, were kept informed on the planned aquifer protection method and on the progress of the studies.

Mr. Ackermann commented favorably on the report with respect to both ground water and surface water aspects and was particularly interested in the possibilities of the Deep Tunnel Project as a key vehicle for regional ground water management. He said[11]:

> "Your proposed design of recharge and observation wells to maintain a positive pressure over the tunnels appears reasonable. Of course, we all recognize that a rigorous program of surveillance will be required, and if local conditions vary from expectations it is conceivable that a few additional recharge wells or increased recharge rates may become necessary.
>
> We visualize that two general plans of management could be developed. One would be a regional one in which a special water district would assume control over all groundwater pumpage, and could thus control water levels over a wide area. Such enabling State laws

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[11] Letter of February 12, 1969, from W.C. Ackermann, Illinois State Water Survey, to V.A. Koelzer, Harza Engineering Company.

> exist, and we would consider this a desirable, and perhaps eventually, an essential system. The other plan of management, which you outline, is to maintain pressures locally through recharge in the vicinity of your proposed works.
>
> The Deep Tunnel, if undertaken primarily for flood and pollution control, will contribute very significantly to the objective of demonstrating that Illinois is taking all feasible and reasonably available means of conserving water. The completion of the Deep Tunnel Project would be the best protection the users of groundwater could have that their groundwater resource will be preserved, because the control of groundwater levels in the areas of the Deep Tunnel facilities which must accompany that Project will fit in admirably with a management program for conservation of the groundwater resource."

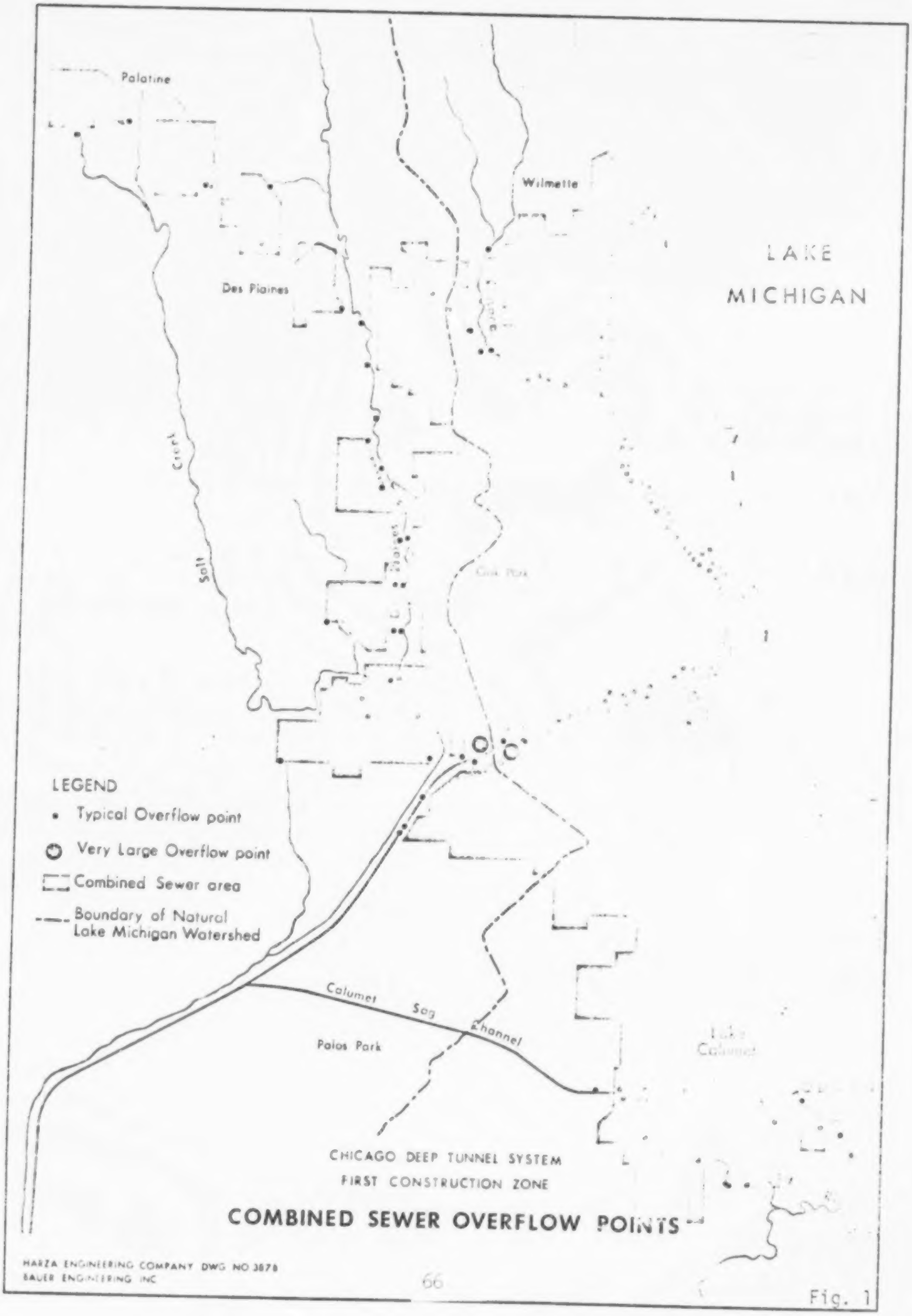
The contacts with the State Board of Health and State Sanitary Water Board were primarily in connection with ground water protection. While specific written comments of individuals in these agencies were not requested, they raised no questions regarding the adequacy of the protective system that has been proposed.

QUESTION: Would you need a new distribution system to effect the recharge?

KOELZER: For the first construction zone, which was about 1/6 of the area, the plan was to have 15 recharge wells. The studies indicated these wells would be adequate to maintain the levels.

QUESTION: In view of the fact that the first portion of storm water contains the highest percentage of the solids, was consideration given to capturing only a first portion of the overflow rather than the total?

KOELZER: This was not done initially for the Deep Tunnel Plan as presented to the Sanitary District, because it was believed that the water quality standards could not be met by capturing only the first flash. This was a criticism that was raised, but information was not then available as to the cost.


Palatine
Wilmette
LAKE MICHIGAN
Des Plaines
Salt Creek
Oak Park
LEGEND
• Typical Overflow point
○ Very Large Overflow point
Combined Sewer area
Boundary of Natural Lake Michigan Watershed
Calumet Sag Channel
Palos Park
Lake Calumet

CHICAGO DEEP TUNNEL SYSTEM
FIRST CONSTRUCTION ZONE

COMBINED SEWER OVERFLOW POINTS

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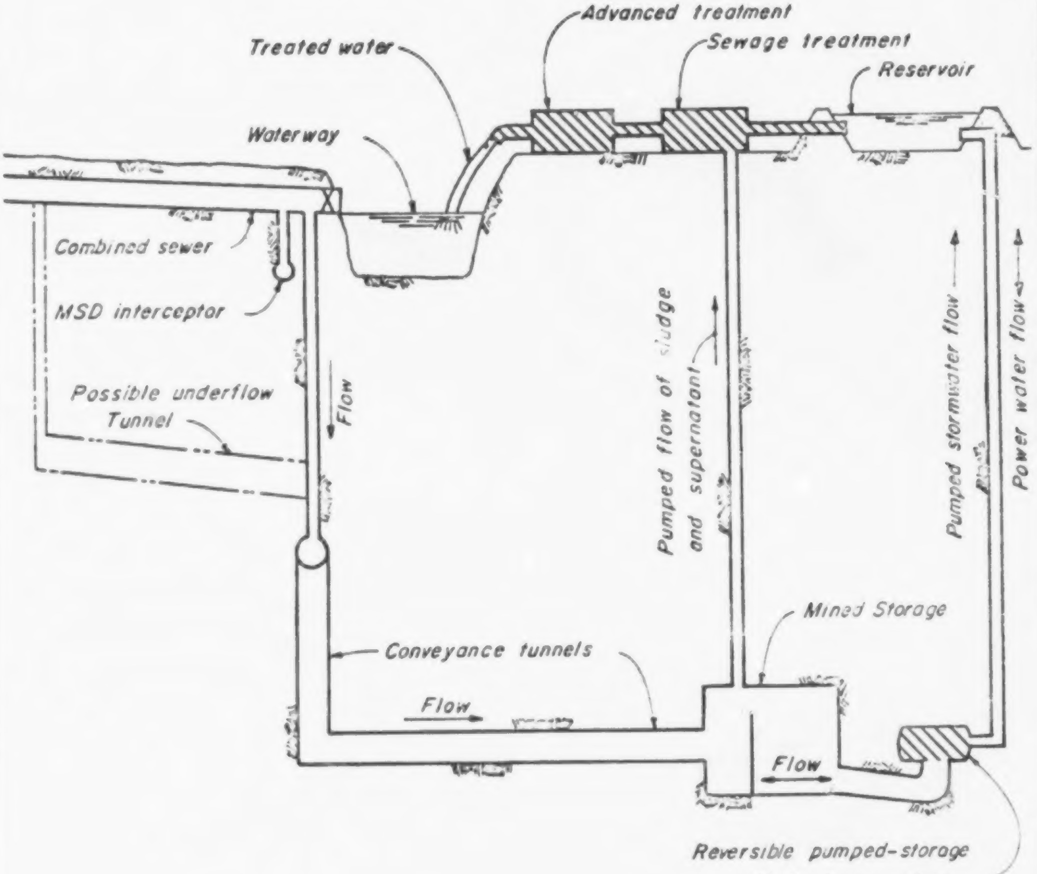
Fig. 1

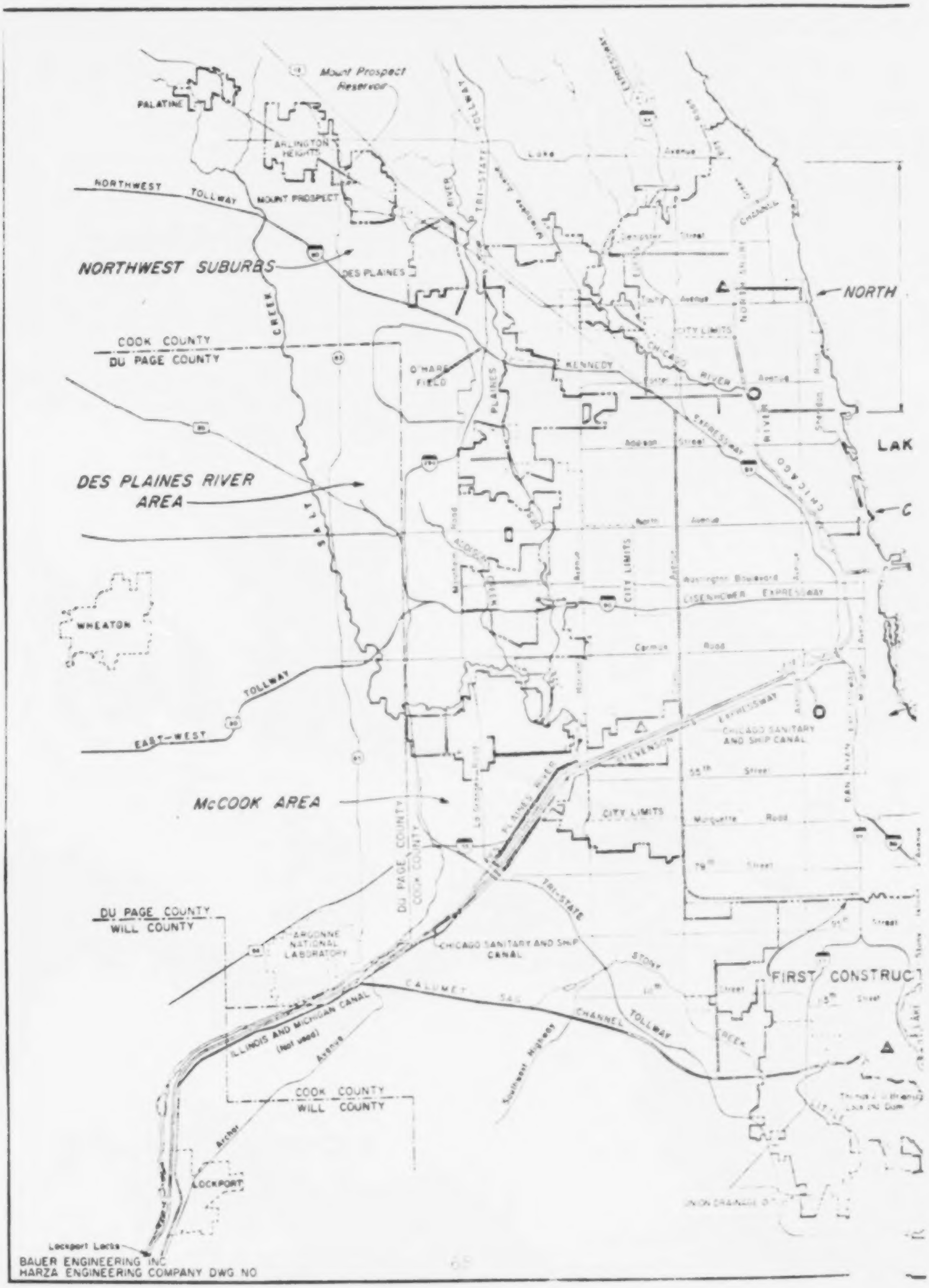

Advanced treatment
Sewage treatment
Treated water
Reservoir
Waterway
Combined sewer
MSD interceptor
Possible underflow Tunnel
Flow
Pumped flow of sludge and supernatant
Pumped stormwater flow
Power water flow
Mined Storage
Conveyance tunnels
Flow
Flow
Reversible pumped-storage Hydroelectric plant

CHICAGO DEEP TUNNEL SYSTEM

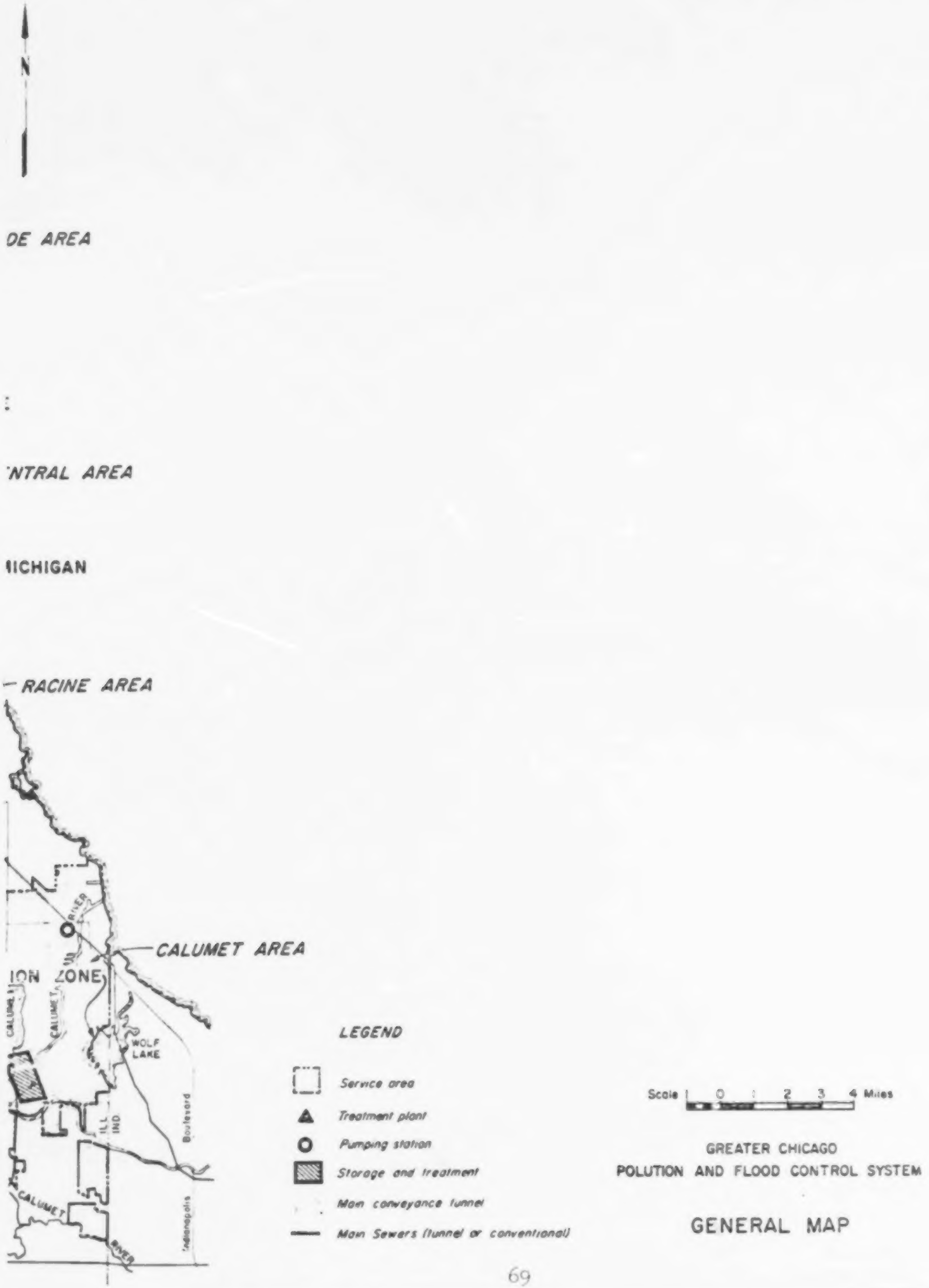
THE DEEP TUNNEL
CONCEPT

HARZA ENGINEERING COMPANY DWG NO. 387 B
BAUER ENGINEERING INC.

Fig. 2


Mount Prospect Reservoir
PALATINE
ARLINGTON HEIGHTS
MOUNT PROSPECT
NORTHWEST TOLLWAY
NORTHWEST SUBURBS
DES PLAINES
TRI-STATE TOLLWAY
DES PLAINES RIVER
Lake Avenue
Dempster Street
Touhy Avenue
NORTH SHORE CHANNEL
NORTH
CITY LIMITS
COOK COUNTY
DU PAGE COUNTY
O'HARE FIELD
KENNEDY
CHICAGO RIVER
Foster Avenue
Addison Street
EXPRESSWAY
LAK
DES PLAINES RIVER AREA
SALT CREEK
North Avenue
WHEATON
Washington Boulevard
EISENHOWER EXPRESSWAY
CITY LIMITS
Cermak Road
EAST-WEST TOLLWAY
STEVENSON EXPRESSWAY
CHICAGO SANITARY AND SHIP CANAL
55th Street
DAN RYAN
McCOOK AREA
CITY LIMITS
Marquette Road
79th Street
DU PAGE COUNTY
COOK COUNTY
DU PAGE COUNTY
WILL COUNTY
TRI-STATE
ARGONNE NATIONAL LABORATORY
CHICAGO SANITARY AND SHIP CANAL
STONY CREEK
FIRST CONSTRUCT
CALUMET SAG CHANNEL
TOLLWAY
ILLINOIS AND MICHIGAN CANAL (Not used)
Archer Avenue
Southwest Highway
COOK COUNTY
WILL COUNTY
Thomas J. O'Brien Lock and Dam
LOCKPORT
UNION DRAINAGE DITCH
Lockport Locks

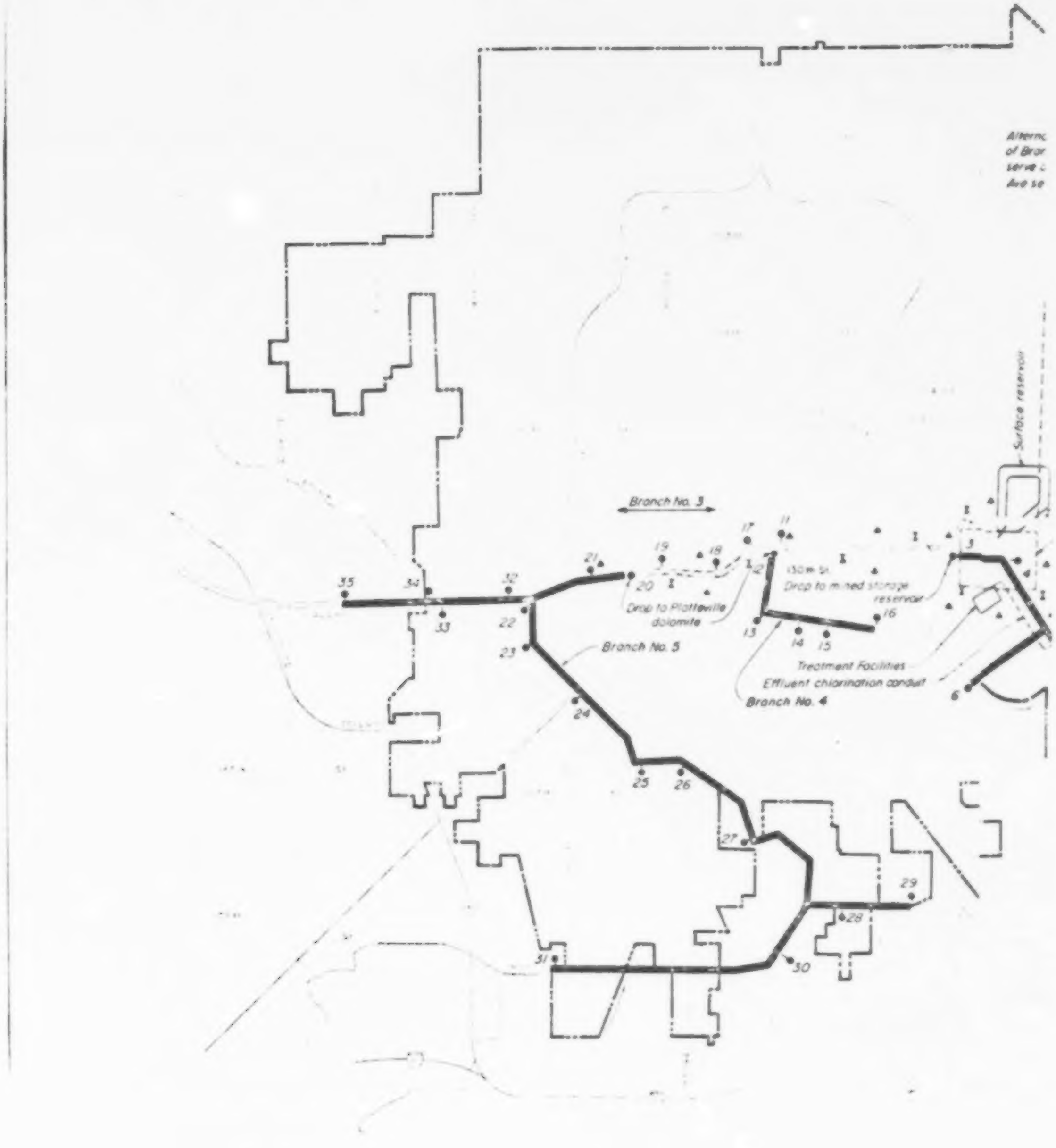
BAUER ENGINEERING INC
HARZA ENGINEERING COMPANY DWG NO


N
DE AREA
:
NTRAL AREA
ICHIGAN
RACINE AREA
RIVER
CALUMET AREA
ION ZONE
CALUMET
CALUMET
WOLF LAKE
ILL
IND
Boulevard
Indianapolis
CALUMET
RIVER
LEGEND
Service area
Treatment plant
Pumping station
Storage and treatment
Main conveyance tunnel
Main Sewers (tunnel or conventional)
Scale 0 1 2 3 4 Miles

GREATER CHICAGO
POLUTION AND FLOOD CONTROL SYSTEM

GENERAL MAP

Fig. 3



Altern
of Bra
serve
Ave se
Surface reservoir
Branch No. 3
35
34
33
32
22
23
21
19
18
17
11
20
12
150th St.
Drop to mined storage reservoir
Drop to Platteville dolomite
3
13
14
15
16
Branch No. 5
Treatment Facilities
Effluent chlorination conduit
Branch No. 4
6
24
25
26
27
28
29
30
31

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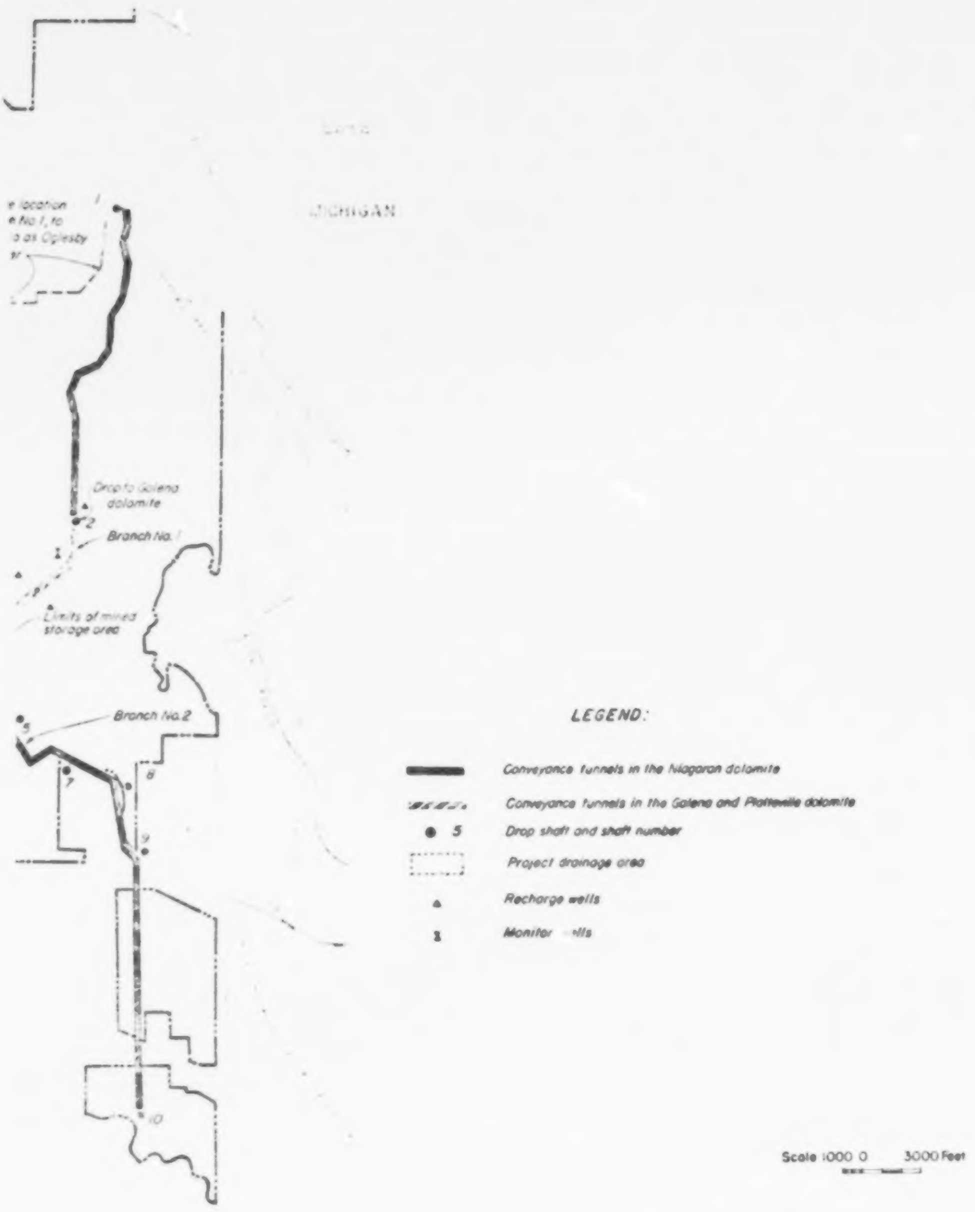

MICHIGAN
e location
e No. 1, to
o as Oglesby
er
1
Drop to Galena dolomite
2
Branch No. 1
Limits of mined storage area
Branch No. 2
5
7
8
9
10
LEGEND:
Conveyance tunnels in the Niagaran dolomite
Conveyance tunnels in the Galena and Platteville dolomite
● 5 Drop shaft and shaft number
Project drainage area
▲ Recharge wells
Monitor wells
Scale 1000 0 3000 Feet

CHICAGOLAND DEEP TUNNEL SYSTEM
FIRST CONSTRUCTION ZONE

GENERAL PLAN

Fig.

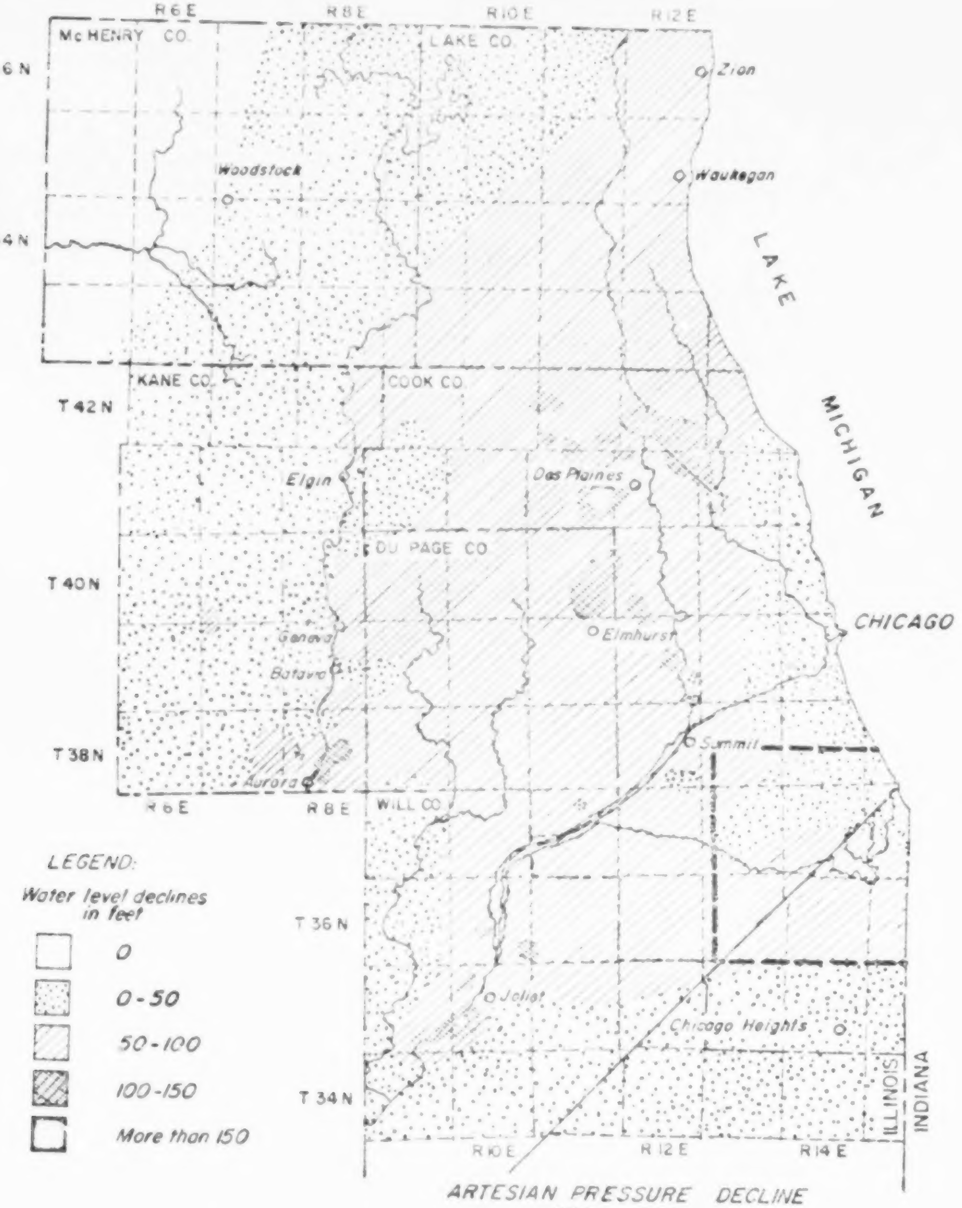
AQUIFER	STRATA	AVERAGE THICKNESS (feet)	PREDOMINANT ROCK TYPE	WATER-YIELDING PROPERTIES	
GLACIAL DRIFT	Pleistocene	65	Till, lenses of sand & gravel	Not tested. Significant deposits of highly productive sands & gravels were not encountered.	
SILURIAN	Niagaran and Alexandrian	400	Dolomitic limestone	Very small yields of water from crevices and solution channels. Yields range from less than 0.1 to 1.0 gpm per foot of drawdown.	
	Maquoketa	170	Shale	Not water yielding, acts as barrier between Silurian & Cambrian-Ordovician aquifers.	
CAMBRIAN - ORDOVICIAN	Galena-Platteville	330	Dolomite	Least permeable unit of the Cambrian-Ordovician aquifer. Yields very small amounts of water from crevices, ranging from less than 0.1 to 1.1 gpm per foot of drawdown.	Most productive aquifer in the region. Yields large amounts of water for municipal and industrial supplies.
	Glenwood-St. Peter	90	Sandstone	Yields small amounts of water.	
	Prairie Du Chien, Eminence and Potosi	340	Dolomite	Locally well creviced central portion of this unit responsible for high yields.	
	Franconia	130	Sandstone	Yields moderate amounts of water.	
	Ironton-Galesville	170	Sandstone	Most dependable and most productive unit of the Cambrian-Ordovician aquifer.	
	Eau Claire	Not penetrated	Shale	Not water yielding, acts as barrier between Cambrian-Ordovician and Mt. Simon aquifers.	
MT SIMON	Mt. Simon	Not penetrated	Sandstone	Not tested. Reported to yield moderate amounts of water.	

NOTE: Water-yielding properties and thickness based on exploration program in First Construction Zone; may be different in other areas.

CHICAGO DEEP TUNNEL SYSTEM
FIRST CONSTRUCTION ZONE

REGIONAL AQUIFER SYSTEMS
Fig. 5

AQUIFERS IN DEEP
TUNNEL ZONE


R6E
R8E
R10E
R12E
McHENRY CO.
LAKE CO.
T 46 N
Zion
Woodstock
Waukegan
T 44 N
LAKE MICHIGAN
KANE CO.
COOK CO.
T 42 N
Elgin
Des Plaines
DU PAGE CO.
T 40 N
CHICAGO
Geneva
Elmhurst
Batavia
T 38 N
Summit
Aurora
R6E
R8E
WILL CO.
LEGEND:
Water level declines in feet
0
0-50
50-100
100-150
More than 150
T 36 N
Joliet
Chicago Heights
T 34 N
ILLINOIS
INDIANA
R10E
R12E
R14E

ARTESIAN PRESSURE DECLINE
1961 - 1966

NOTES:

1. The piezometric surface represents the water pressu
Aquifer, i.e. the level to which water will rise in a w

2. Drawings based on Illinois State Water Survey; Sas

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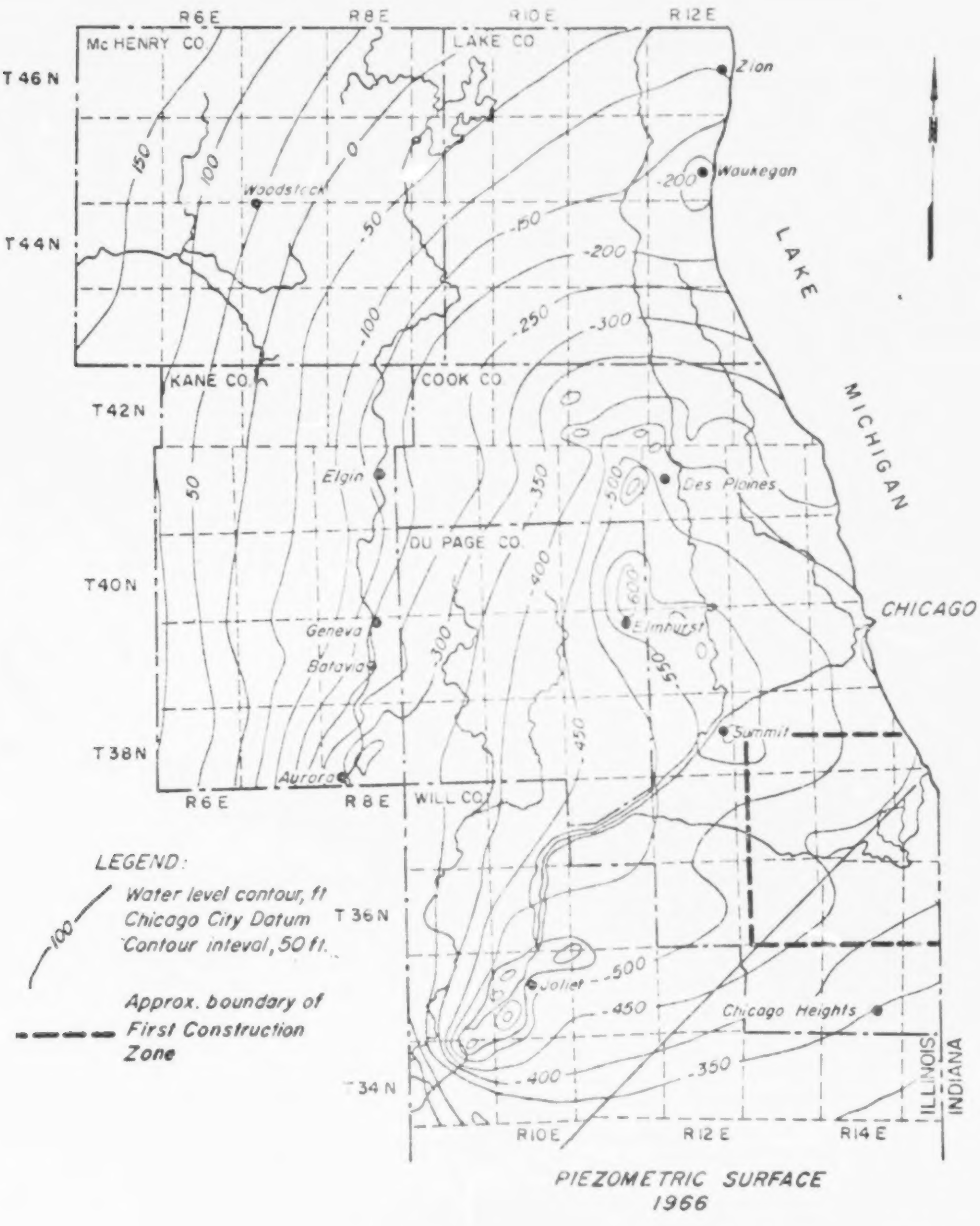

R6E
R8E
R10E
R12E
McHENRY CO.
LAKE CO.
T46N
T44N
T42N
T40N
T38N
T36N
T34N
Zion
Waukegan
Woodstock
LAKE MICHIGAN
KANE CO.
COOK CO.
DU PAGE CO.
WILL CO.
Elgin
Des Plaines
CHICAGO
Geneva
Batavia
Elmhurst
Summit
Aurora
Joliet
Chicago Heights
ILLINOIS
INDIANA
R14E
150
100
50
0
-50
-100
-150
-200
-250
-300
-350
-400
-450
-500
-550
-600
LEGEND:
Water level contour, ft Chicago City Datum Contour inteval, 50 ft.
Approx. boundary of First Construction Zone

PIEZOMETRIC SURFACE
1966

Scale 0 5 10 Miles

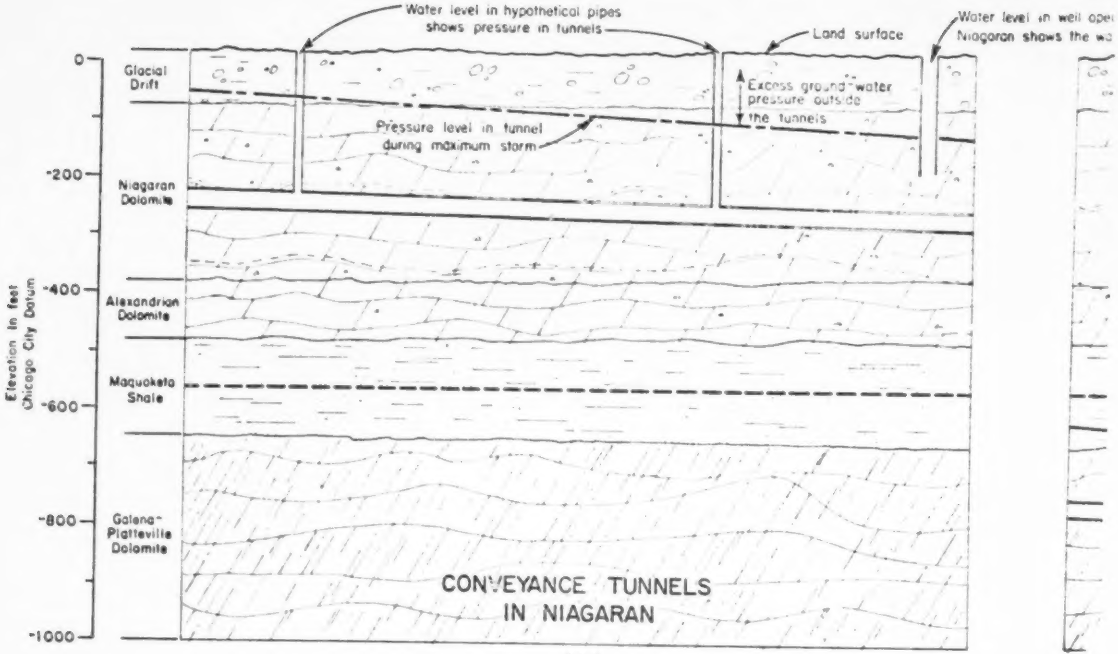
level in the confined Cambrian-Ordovician
penetrating the artesian aqifer.

nan, McDonald and Randall, 1967

CHICAGOLAND DEEP TUNNEL SYSTEM
FIRST CONSTRUCTION ZONE

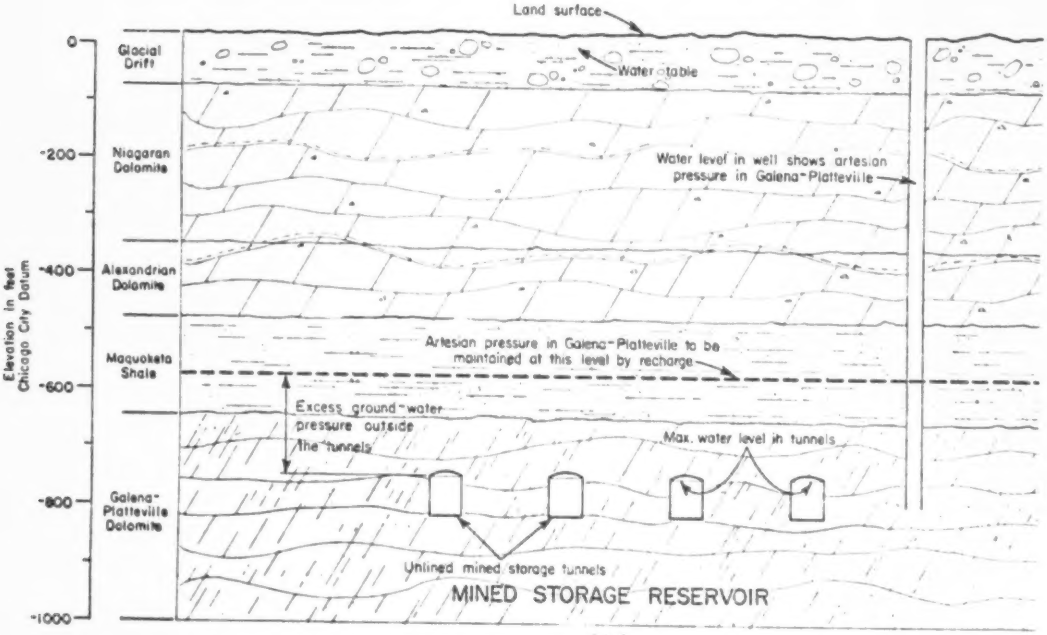
CAMBRIAN-ORDOVICIAN AQUIFER
WATER LEVEL MAPS

Fig. 8



Water level in hypothetical pipes shows pressure in tunnels
Land surface
Water level in well ope
Niagaran shows the wa
Excess ground-water pressure outside the tunnels
Pressure level in tunnel during maximum storm
0
-200
-400
-600
-800
-1000
Elevation in feet Chicago City Datum
Glacial Drift
Niagaran Dolomite
Alexandrian Dolomite
Maquoketa Shale
Galena-Platteville Dolomite
CONVEYANCE TUNNELS IN NIAGARAN

(A)



Land surface
Water table
Water level in well shows artesian pressure in Galena-Platteville
Artesian pressure in Galena-Platteville to be maintained at this level by recharge
Excess ground-water pressure outside the tunnels
Max. water level in tunnels
Unlined mined storage tunnels
0
-200
-400
-600
-800
-1000
Elevation in feet Chicago City Datum
Glacial Drift
Niagaran Dolomite
Alexandrian Dolomite
Maquoketa Shale
Galena-Platteville Dolomite
MINED STORAGE RESERVOIR

(C)

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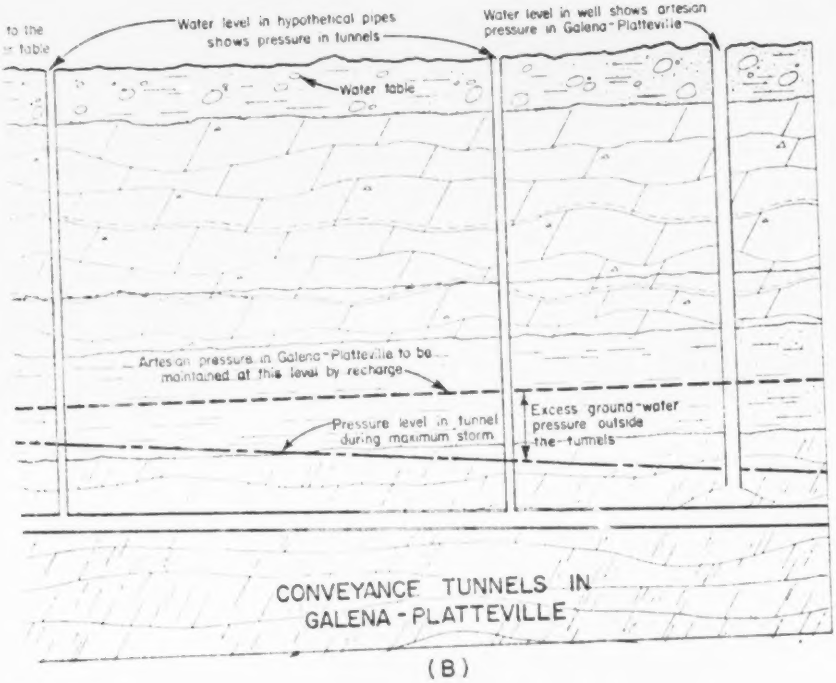
to the
table
Water level in hypothetical pipes shows pressure in tunnels
Water level in well shows artesian pressure in Galena-Platteville
Water table
Artesian pressure in Galena-Platteville to be maintained at this level by recharge
Excess ground-water pressure outside the tunnels
Pressure level in tunnel during maximum storm
CONVEYANCE TUNNELS IN GALENA-PLATTEVILLE

(B)

EXPLANATION:

Arrow indicates direction of water pressure.

CHICAGOLAND DEEP TUNNEL SYSTEM
FIRST CONSTRUCTION ZONE

PRINCIPLE
OF AQUIFER PROTECTION

Section 5

THE POTENTIAL OF PUMPED STORAGE FOR HYDROELECTRIC GENERATION IN MULTI-LEVEL DEEP TUNNEL SYSTEMS

by

Kenneth E. Sorenson
Vice-President
Harza Engineering Company
400 West Madison Street
Chicago, Illinois 60606

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MULTIPLE USE OF UNDERGROUND RESERVOIRS FOR POWER GENERATION

USES OF RESERVOIRS

Large underground chambers excavated for urban flood control would have relatively infrequent use, and would very rarely be fully used as single-purpose reservoirs. One possible dual function could be for hydroelectric pumped-storage. Another multi-purpose function could be the circulation of condenser water for underground nuclear generating plants.

This paper describes briefly the possibilities and implications of such multi-purpose uses.

PROBLEMS OF THE POWER INDUSTRY

From the viewpoint of transmission and reliability, generation is preferred close to the major urban centers. Obtaining publicly acceptable routes for high voltage lines from more distrant plants is becoming increasingly difficult.

In the case of pumped-storage, many large urban areas do not have favorable topography for conventional hill-top reservoirs. And yet, the functions of such plants for day time peaking and system reserves are best served if close to the load centers. Even where suitable sites are near by, preservationist opposition arises, as with Storm King Mountain near New York City. Underground installations could overcome both the topographical and preservationist obstacles. Nuclear plants also present problems for urban or suburban siting, in some cases due to unwarranted fear by the public of accidental radioactive emissions. Warm water discharges from condensers also have led to opposition. Underground placement of nuclear plants can give assurances of isolation during accidents and greater protection against sabotage. Surface reservoirs of a flood control and pollution abatement scheme might possibly serve also as cooling ponds.

DESCRIPTION OF POWER INSTALLATIONS

A simplified section through an underground pumped-storage and nuclear power project is shown on Figure 1. The principal elements are:

1. An upper reservoir, lake or ocean.
2. An upper intake and discharge structure, and access building.

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3. Vertical shafts for pentrocks, for access and cables, and for chamber construction and air vent.
4. Lower level equipment chambers for reversible pump-turbines, electrical equipment, and the possible nuclear reactors and turbine-generators.
5. A lower reservoir serving the pumped-storage project and the possible nuclear plant condensers.

If not part of a flood control scheme, the lower reservoir of such a power project needs to be from 2,000 to 3,000 ft. deep to be economical. If the excavated reservoir chamber is essential for other than power use (e.g. flood control), the pumped-storage and nuclear generating plant would be economical at much lesser depths. A plan of the lower level chamber for the power development alone is shown on Figure 2. This arrangement provides for cooling water flow into the condensers and pumping of heated water to the surface reservoir.

Both in section and in plan, the power development can be adapted to a multi-purpose flood control and pollution abatement scheme. The large pumping capacity of the reversible pump-turbines would permit rapid evacuation of the lower reservoir after a flood. In a single-purpose flood control scheme, equivalent large pumps would be very expensive for their very infrequent use.

FUNCTION OF PUMPED-STORAGE

The uses of pumped-storage projects are so generally well known that only brief mention is needed here. The major functions are:

1. Load center reserve.
2. Short-term peaking.
3. Load regulation.
4. Energy economy.

As reserves, such plants can be a source of start-up power for thermal plants, as was needed in the 1965 Northeast blackout.

For short term peaking and load regulation, pumped-storage plants offer a faster response to load changes and greater efficiency under variable loads. An example of this type of operation is shown on Figure 3.

As more and larger fossil and nuclear fueled plants are installed, advantages arise in the use of pumped-storage to conserve low-cost, off-peak energy for on-peak use, with a resulting energy economy. An example of this type of operation is shown on Figure 4.

NUCLEAR POWERPLANT COMBINATION

The use of an underground reservoir for condenser water at a nuclear plant creates complications in the configuration of the lower chamber and in pumped-storage operation. On the other hand, the access shafts and other power plant facilities of a pumped-storage project could serve an economical dual function for the nuclear plant. Greater public acceptance and the economic advantages of urban siting would be the principal determining factors for inclusion of nuclear plants in the multi-purpose scheme.

FLOOD CONTROL AND POWER OPERATIONS

In any multi-purpose reservoir one function takes priority, or a compromise in functional uses is established. In the case of Lake Mead behind Hoover Dam, compromises are made in operation, but generally irrigation and municipal water supply have priority over power generation. There have been years when so-called "prime energy" from the Hoover powerplant has been only 60% of the contract amounts.

All generating plants, whether hydroelectric or thermal-electric do not offer 100% reliability. For the former, forced outages of equipment are rare, but extreme low-water conditions can curtail output. Fossil-fuel plants have more frequent equipment outages and occasional fuel shortages. Nuclear plants have experienced the greatest equipment failures, but no lack of fuel.

Multiple use of underground reservoirs for flood control and pumped-storage would create some conflict in uses, and cause partial or total reduction in generation. However, the infrequent use of the full volume for floods would cause much less outage of generation than is normal for other types of plants. In a study made for the Chicago Area, reductions in generation over a 96-year historic period would have occurred less than 0.1% of the operating time.

One example of the combined use of lower and upper reservoirs for flood control and power generation during a storm is shown on Figure 5. There is indicated for this rather severe storm some curtailment in generation and a small amount of overflow of untreated sewage to the waterway. Neither the curtailment nor the overflow are significant.

COSTS

Any flood control and pollution abatement scheme must be accepted by the community on the basis of its costs versus the economic and intangible benefits. The addition of pumped-storage generation can contribute a commercial revenue to the multi-purpose scheme that exceeds the incremental cost of the power features.

Single purpose pumped-storage projects in the U. S. now have a construction cost of $100 to $150/kilowatt. The incremental constructions cost of the pump-turbine installation in a flood control scheme is about $75/kilowatt. In the case of the Chicago Regional Plan, about 20% of the cost of the single-purpose, flood control works could be carried by the hydroelectric generating plant.

No estimate has been made of the costs or benefits from underground nuclear installation in this type of multi-purpose development.

INSTITUTIONAL PROBLEMS

It is relatively easy to determine the technical feasibility and economic value of the addition of generating facilities in a flood control and pollution abatement scheme. The institutional and legal aspects create substantial obstacles.

Agencies in charge of flood control and/or sewage treatment do not usually have the capability nor the legal powers to enter into the electric power business. On the other hand, the utility companies have an obligation to meet the growing power demands of their service area, and cannot rely on the uncertain actions of other agencies.

Because of costs, it is not feasible for a utility company to construct an underground pumped-storage project that could serve for flood control. Agencies empowered to construct underground flood control reservoirs are dependent upon federal, state and local legislatures for the funding and timing of their projects. This makes it most difficult for a utility company to guarantee, on an equity or purchase basis, that the supplemental pumped-storage power can be absorbed by the urban electrical system at a fixed price.

Despite these problems, there are precedents in the U. S. for mixed public and private cooperation in multi-purpose development. The successful examples have had the benefit of properly written legal charters and strong political support.

CONCLUSION

In today's complex society, the many needs of our urban communities and the possible beneficial developments must be considered from a multi-functional viewpoint. The theme of this seminar is the use of our underground urban potential for combined sewer overflow and flooding problems. Both hydroelectric and nuclear power plants can be valuable complementary functions.

QUESTION: Did you make any estimate of the amount of heat that could be dissipated through this underground system?

SORENSON: There would be no heat dissipated as far as the underground installation itself is concerned. The only way it could be dissipated is on the surface through such things as cooling ponds.

QUESTION: Is pumped storage an essential part of the Deep Tunnel Plan from an economic standpoint?

SORENSON: It is not essential, but it contributes through commercial revenue and through the large capacity pumps which could not be considered in a single purpose reservoir.

QUESTION: What is the overall electrical and mechanical efficiency of the system?

SORENSON: The hydroelectric equipment is very efficient. Turbines are generally around 90% and generators around 97%. The use of energy, taking off-peak energy and converting to on-peak energy, requires an input of about 1.4 kw-hrs for every 1.0 kw-hr received. The relationship is the same for both the hill top and underground storage.

QUESTION: How much could the required nuclear power capacity be reduced if this were adopted?

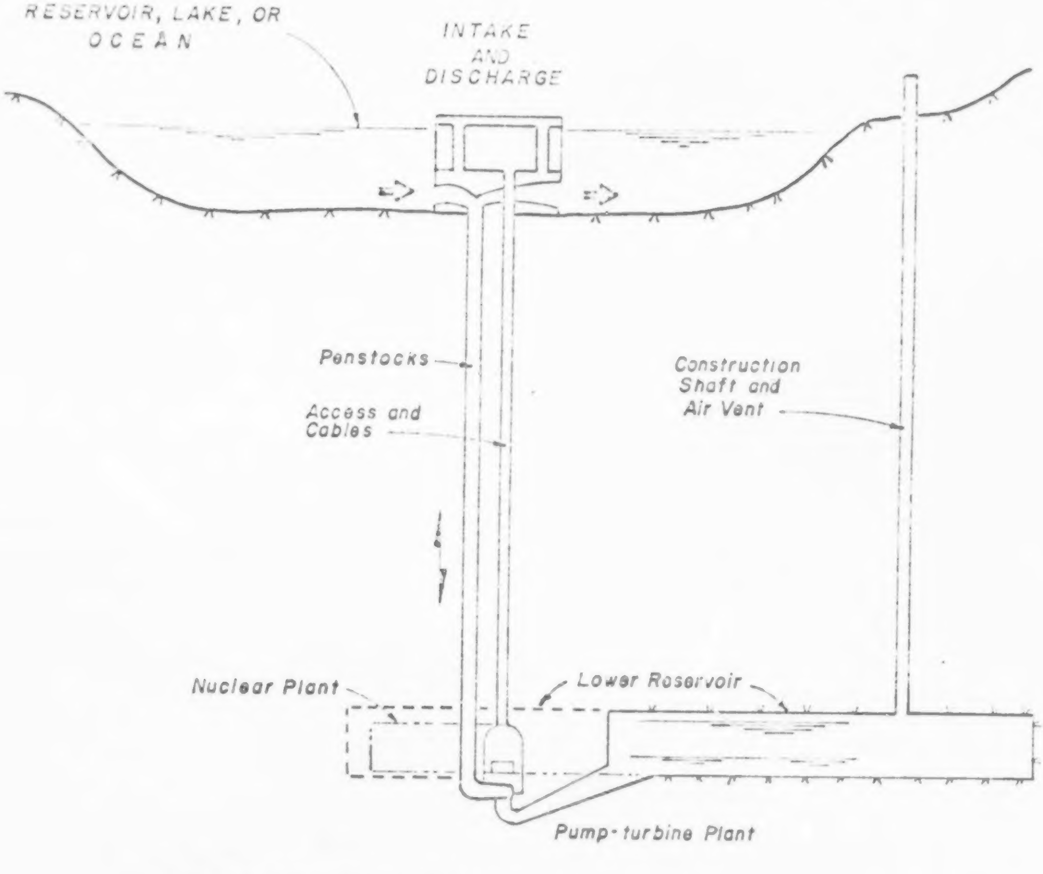
SORENSON They do not replace nuclear plants. The two are complementary. There is more total energy required in a combination pumped storage with nuclear or fossil fuel plants.

QUESTION: If pumped storage replaced older less efficient steam plants, would the overall heat loss still be the same?

SORENSON: There would be an offsetting effect.

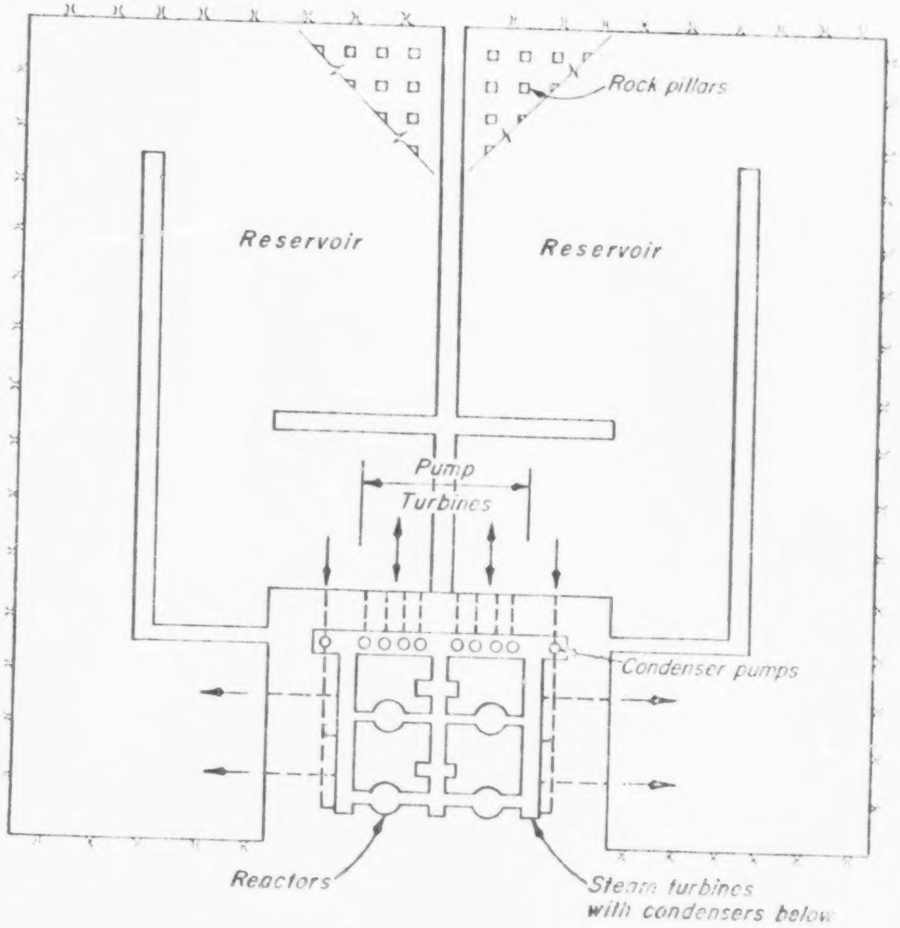
QUESTION: Are there any problems involved with using sewage in the generators?

SORENSON: No, we do not anticipate problems.


RESERVOIR, LAKE, OR OCEAN
INTAKE AND DISCHARGE
Penstocks
Access and Cables
Construction Shaft and Air Vent
Nuclear Plant
Lower Reservoir
Pump-turbine Plant

UNDERGROUND PUMPED-STORAGE
SECTION

(FIGURE 1)


Rock pillars
Reservoir
Reservoir
Pump
Turbines
Condenser pumps
Reactors
Steam turbines with condensers below

PUMPED STORAGE-NUCLEAR PLANTS
LOWER LEVEL PLAN

(FIGURE 2)

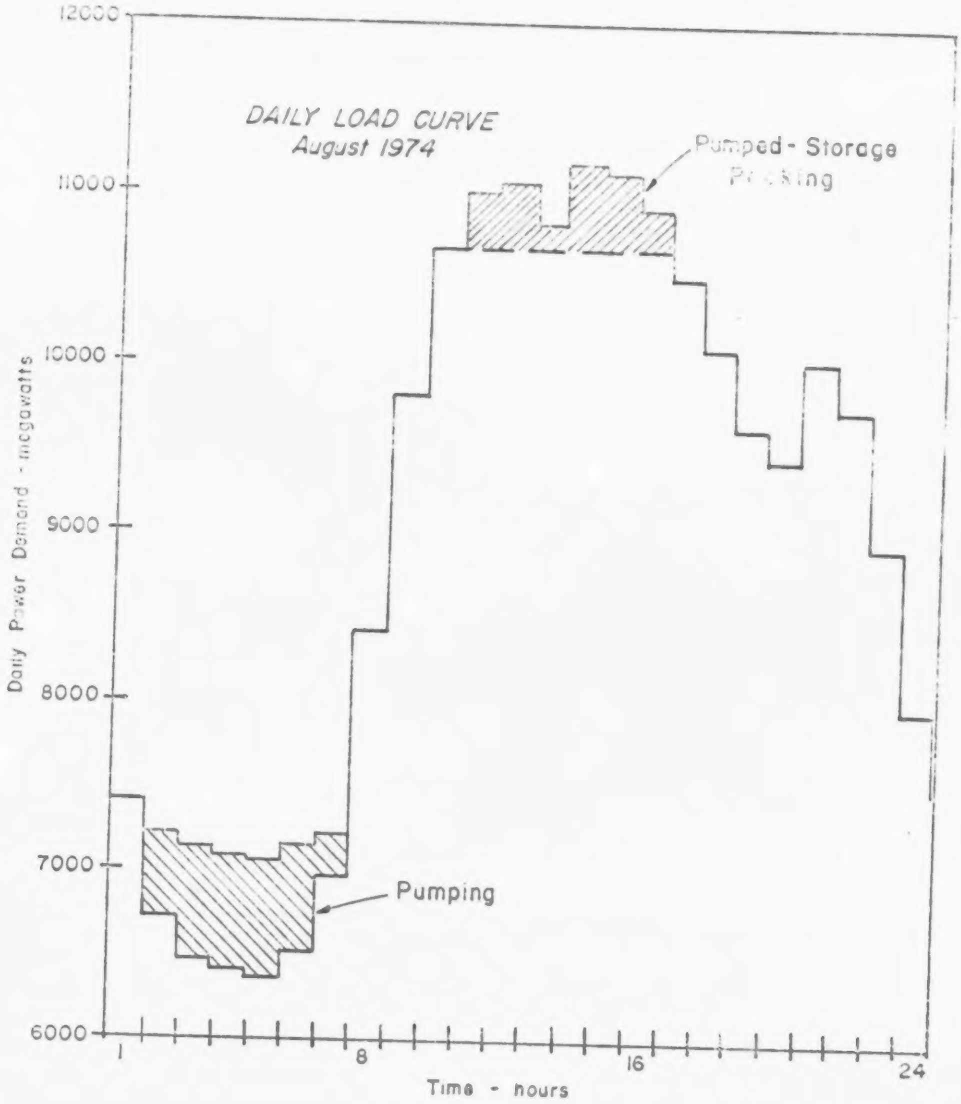
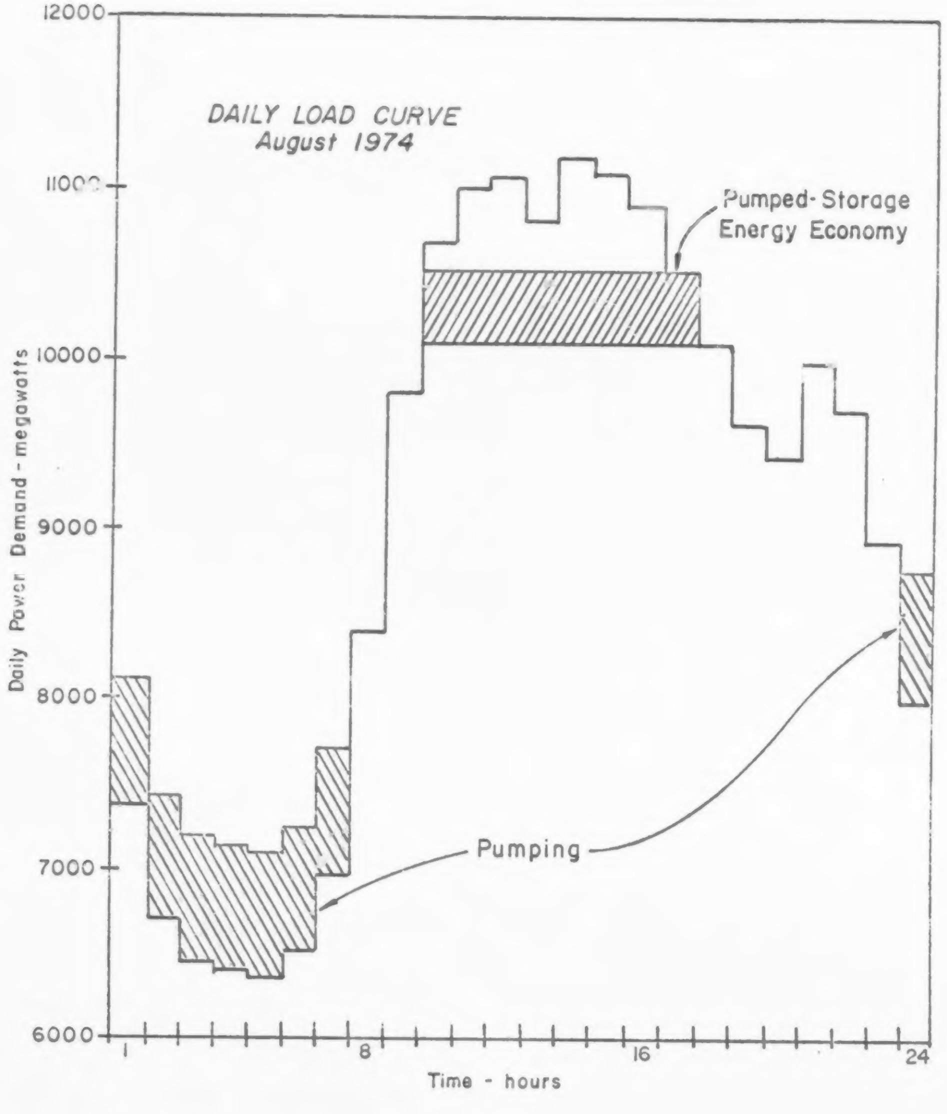
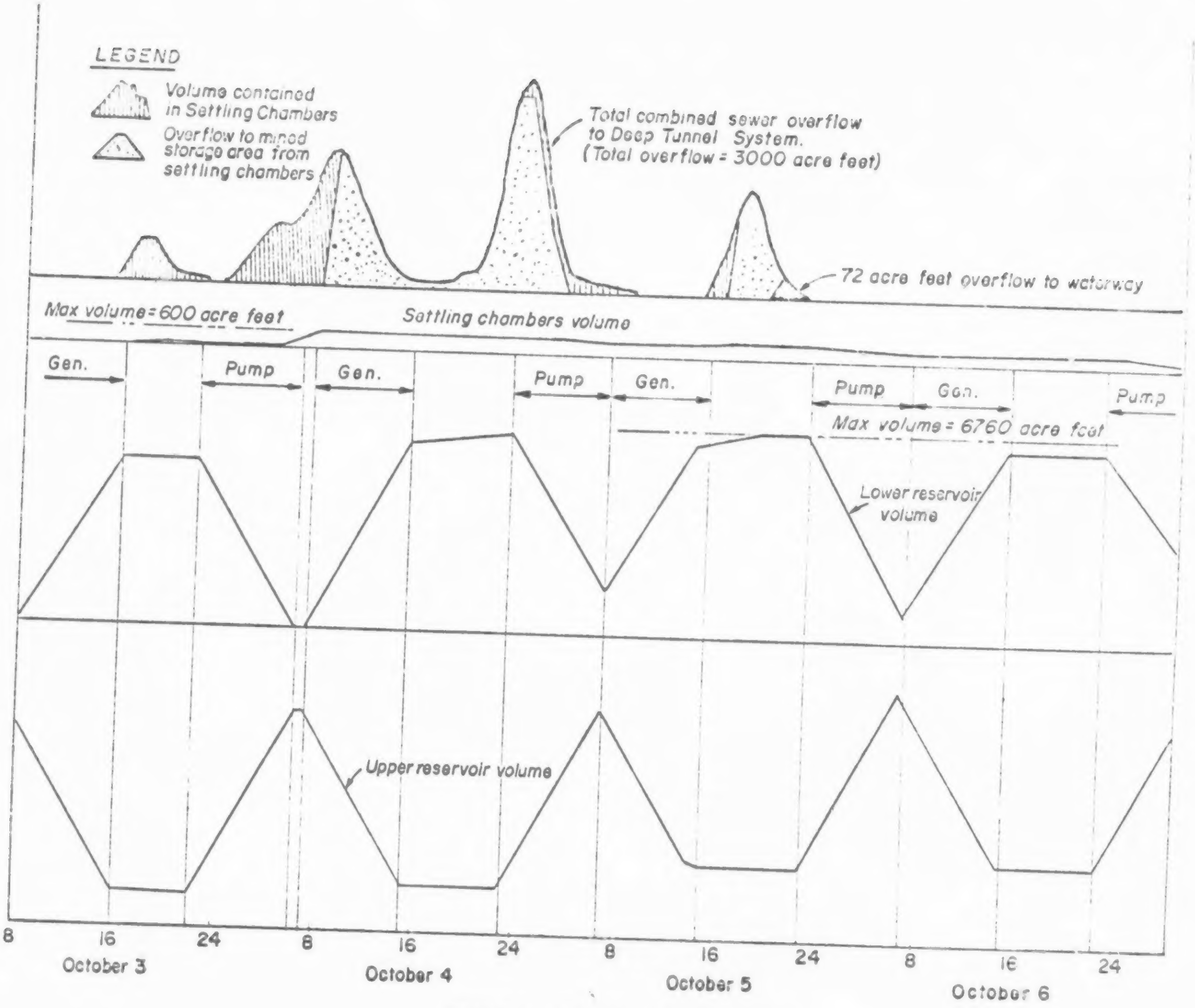

DAILY LOAD CURVE
August 1974
Pumped-Storage
Peaking
Pumping
12000
11000
10000
9000
8000
7000
6000
Daily Power Demand - megawatts
1
8
16
24
Time - hours

FIGURE 3



DAILY LOAD CURVE
August 1974
12000
11000
10000
9000
8000
7000
6000
Daily Power Demand - megawatts
Pumped-Storage
Energy Economy
Pumping
1
8
16
24
Time - hours

FIGURE 4



STORM OF OCTOBER 3, 1955

(FIGURE 5)

Session 3

EXPERIENCES WITH HARD ROCK TUNNELING AND MECHANICAL MOLES

Moderator - W. T. Painter

Section 6

EUROPEAN DEVELOPMENT AND EXPERIENCE WITH MECHANICAL MOLES IN HARD ROCK TUNNELING

by

Pieter Barendsen
Chief Engineer, Product Development
Atlas Copco MCT AB
Stockholm, Sweden

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INTRODUCTION

The subject we are dealing with during the first part of this morning's session - i.e., "European Development etc." - might be construed to indicate the existence of a certain rivalry in the development of full face boring equipment between Europe and the US.

A historical review too may contribute towards such an impression. On 24th March, 1853 A. F. Edwards, the man who had planned and estimated the cost of the Hoosac Railway Tunnel to the last dollar - $1,948,557 no more and no less - reported: "The first model of Wilson's patented stone-cutting machine for tunnel excavation in rock is now at the Hoosac Mountain. The result of its working in the natural rock has been from 14 to 24 in./hr., on a full circle of 24 ft. diameter." From this he proceeded to line up a detailed working schedule, substantiated by calculations showing that with two machines one at each end, the entire excavation would take exactly 1,005 days.

The Wilson machine seems to have driven a total distance of 10 ft. before being consigned to the scrap-heap, a fate shared by two further machines, attempting to cut diameters of 17 and 8 ft. respectively, in the same tunnel. Subsequent events showed that Edwards had gravely underestimated the technical difficulties in general of this tunnelling project: Hoosac, with a total length of 24,416 ft., took just under 20 years to complete and the costs totalled some $10 million. And, while there exists no doubt that out of the Hoosac mess ascended the American compressed-air industry which took world - leadership in developing and providing the mining and construction industry with the early machines and tools for the mechanization of underground rock excavation work, it is equally clear that technological developments at that time just had not advanced far enough to create workable full face boring machinery.

Knowing what we do today about the problems of tunnelling by mole, the results reached as early as 1884 with Col. Beaumont's 7 ft. diameter machines - the first to go on record as being successful in hard rock - are really quite impressive: one of these drove about 115 ft./week in sandstone in the first Mersey tunnel in England, while the other bored a total distance of 8,400 ft. in chalk in a pilot tunnel under the Channel, maintaining an average speed of 50 ft./day for not less than 53 working days at a stretch. The machines worked with kerfing tools on a rotating head and were powered by compressed air. (Fig. 1)

In the years between 1884 and 1953 a dozen machines were designed and tested, most of them in Europe, but none of them really progressed beyond the prototype stage.

All of the manufacturers presently engaged in the full face boring business - my own firm being one of them - will readily admit that the US firm of James S. Robbins and Associates did most of the

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pioneering work on the present generation of moles. Development work began in 1953 and a first machine, still partly equipped with kerfing tools, was put to work in 1954, reaching an advance of some 10 ft./hr. in interbedded sandstone, limestone and shale with compressive strengths of up to 27,000 psi.

Today still, the majority of moles in use are of US design, even if some of them have been built under license outside this country.

ROCK HARDNESS

Before we investigate in which way modern full face boring development in Europe deviates from US praxis, I would like to pause for a moment and look at the meaning of the term "Hard Rock" which occurs in the titles of quite a few of the presentations made during these two days.

According to international contracting practice, all material occurring within the earth's crust is regarded as rock, if it is so hard that drilling and blasting or some similar, high-energy process must be used to break it up.

It has, unfortunately, become customary to relate the rock's hardness - i.e., its resistance to boring - to its compressive strength, expressed in psi, while a suitable expression for the cohesive strength of the rock is what is really relevant.

When we consider the values of compressive strength quoted in literature for some of the more common types of rock (Fig. 2), it is evident that different rocks vary considerably in hardness and that considerable spread exists for rocks of the same type.

This not only depends on the fact that there is a large variation for rock types with the same petrographical designation in different parts of the world, but also on the actual testing procedure used.

In publications dealing with full face boring we often find one single maximum value quoted for the compressive strength of the rock, while nothing is said about how this value was obtained or what proportion of the total volume of rock is represented by the sample quoted. This is very unsatisfactory. To compare the "borability" of different types of rock in a meaningful way the following factors should be taken into consideration:

1. The compressive and the shear strengths of samples of fresh, homogeneous and non-fissured rock. The shape and size of the sample and, in the case of bedded rock, its orientation as well as the testing procedure, method of preparation, moisture content, loading rate and number of measurements, should be regulated internationally or at least be specified for each individual test.

2. The Mohs' hardness or some other comparative hardness value for the minerals in the sample, together with the grain size and the distribution of mineral constituents.

3. Fissures, cleavage planes and other discontinuities in the ranges of 0.4 to 2.0 in. and 2 to 20 in.

4. The abrasiveness of the rock, expressed by the volumetric percentage of quartz and felspars, or by a standardized abrasion test.
5. The porosity of the rock as well as the matrix material between individual crystals or mineral grains.

In the case of a tunnel driven at great depth, rock pressure should also be taken into account, as this may be a factor which could make the rock easier or harder to bore.

In spite of the imperfections of the compressive strength as the sole yardstick for measuring the borability of a rock, it is widely used, mainly because it is a value which can be established rapidly from small samples such as diamond-drilled cores. It would seem to be of the greatest importance that the authorities responsible for the design and construction of tunnels and other underground openings should improve substantially the quantity and the quality of the information made available at an early stage to those who will have to carry out the actual excavation. The argument that such pre-tendering investigations would raise the cost of the completed tunnel is not valid. On the contrary, the increase in cost would be more than offset because the contractors would not feel obliged to load their bids so as to safeguard themselves against the financial consequences of unknown and unfavourable rock conditions.

For the purpose of further discussion I think that, no matter what each of us sees as the most important factor to describe "the hardness of the rock," we can agree upon such a definition of "hard rock tunnelling" that it automatically excludes the use of shields for the support of unstable or running ground.

DIAMETER RANGES FOR HARD ROCK TUNNELLING MACHINES

As moles usually are designed for specific tunnel diameters, we might review what range one is nowadays attempting to cover.

As we know, tunnels are driven for many different purposes and with widely varying cross-sectional areas. (Fig. 3)

Some of them have such small diameters that they should rather be considered as underground conduits and these are usually constructed in soil or very soft rock formations by means of augering or tubepressing.

A diameter of 6 ft. must be considered as the lower limit for excavation by explosives or by means of a full face boring machine operating inside the tunnel and most civil engineering experts tend to regard 20 ft. as the approximate upper limit for economical boring operations under normal conditions today. Special circumstances, such as the presence of easily bored, weak rock which would need elaborate support if blasted or when tunnelling is done close to existing buildings or under water, have sometimes established their own scale of economic value and led to the use of machines of some 30 ft. in diameter.

Examples of such very large machines are the Robbins mole used at the Mangla Dam, later rebuilt and now in use at the second roadway tunnel underneath the Mersey; the Robbins machine used on the Paris subway and, more recently, the Robbins mole for the Heitersberg Railway tunnel in Switzerland and the Wirth two-stage machine (25 and 35 ft. diameter) for a roadway tunnel near Lucerne in that same country, reaming from a 12 ft. diameter pilot bore.

Generally speaking, one may conclude that within the field from 6 ft. to 20 ft. diameter the choice of excavation method applied is dependent on the quality of the rock and economic conditions rather than on the diameter.

VARIOUS TYPES OF EUROPEAN MOLES

The rock boring machines available for tunnelling in Europe today, may be divided into two groups, according to the method of operation: Machines that work the full face of the tunnel at any moment, while being advanced continuously along the tunnel axis.

This is the type of machine which, in this country, is usually described by the term "mole". Presently, three manufacturers in Europe offer this type of equipment: Wirth and Demag in Germany and Atlas Copco of Sweden through their subsidiary in Switzerland. (Fig. 4)

All of these machines bore tunnels with a circular cross section, because the cutter-or boring-head is rotated around an axis that coincides with that of the tunnel itself. Diameters for which standard machine designs are available more or less "off the shelf," range from just under 9 ft. to just over 14 ft. In this respect Europe seems to follow normal US trends.

Machines with one or more cutter heads of dimensions substantially smaller than the tunnel cross section which work the face by a combined rotating and sweeping movement and are advanced stepwise in the longitudinal direction of the tunnel. (Fig. 5)
Such machines can, because of their design, cut a tunnel of non-circular cross-section and are, therefore, of special interest in mining operations where a flat footwall is required for haulage purposes. The majority of these machines are equipped with "pick-

type" tools and have not been designed to work rock any harder than the relatively soft formations encountered in coal mining. Well known manufacturers in the field are: Mayor & Coulson, Greenside-McAlpine, Bretby and Dosco in England, Eickhoff and Demag in Germany and Alpine in Austria. A number of machines for use in soft materials like coal, gypsum and salt have also been developed in the USSR, from where they have spread to other countries behind the Iron Curtain.

Recently Atlas Copco have entered this field with designs suitable for work in hard rock. These machines use the same type of cutters and cutter heads as are used on the standard machines for circular cross sections produced by this firm, but in different configurations and with a different pattern of movement in space.

CUTTING TOOLS

One of the main problems in tunnelling without the use of explosives lies in the development of tools which, at an economically acceptable level, are capable of continuous breakage of the rock, resulting in a fragmentation suitable for a smooth, uninterrupted transportation of the muck away from the tunnel face.

Tools employed on tunnelling machines have, so far, always been of the mechanical kind which break up the rock by a crushing and shearing action. Other methods of rock breaking are possible but, even if some of them have reached the laboratory testing stage, nearly all of them are still too "exotic" to be of any practical use in the immediate future.

One must admit, however, that some highly interesting results are beginning to be reported from this country on the use of high pressure water jets for the destruction of rock by erosion, a field pioneered in Europe (USSR and Great Britain).

Mechanical breakage of rock is effected by inducing stresses exceeding its compressive/shear strength by loading it with a wedge or cone-shaped tool until cracks are formed and chips are loosened. In order to allow the tools to act upon the rock face continuously, they are commonly shaped as rotating bodies, "roller bits" or "disc cutters," spinning freely and mounted on a revolving boring head of tunnel diameter. (Fig. 6a, b). The thrust of the tools against the face is generally exerted in a direction which is parallel to the tunnel axis.

The roller bit, which was inherited from the oil industry, is the older of the two and has so far predominated for work in hard rock. The disc cutter, because of its generally somewhat lower cost per foot of tunnel and smaller production of fines, is fast making inroads, even for work in hard rock like granite and gneiss.

The rock may also be worked by fixed cutters such as "picks" or "tips" which are moved along a linear or curved path to cut a groove in the tunnel face. (Fig. 6c) These tools are usually arranged so that the main cutting forces occur in a plane at right angles to the tunnel axis. An important consequence of this arrangement is that machines in which such tools are employed do not require such high thrusts against the face as do tunnelling machines in which rotary cutters are used.

Possibly influenced by early experience with cutting or "ripping" tools in coal mining, many have considered them less suitable than roller cutters for work in harder rock. During recent years this opinion has been proved wrong. (Fig. 7)

At the 1968 Tunnelling and Shaft Sinking Conference held in Minneapolis by the University of Minnesota, Dr. Nevil Cook of South Africa indicated what requirements of cutting geometry have to be met for the cutting of hard rock. The fact that these are not purely theoretical observations has been proved by recent Rock Cutter field tests carried out by the Chamber of Mines of South Africa in order to arrive at a non-blasting stopping method for the deep lying gold mines in that country.

Figures are available from Switzerland too which show that this kind of tool is suitable for the cutting of hard and abrasive rock - a quartzitic sandstone with a compressive strength varying from 25,500 to 34,000 psi and with a quartz content of not less than 60 per cent - while the total tool costs for the tunnelling machine in question, including the cost for renovation of the tool holders, were slightly below the costs for drill steel, explosives and blasting caps in that part of the same tunnel which was driven by the conventional method. (Julia hydro electric power station, 1967/68)

While on the subject of cutting tools, it may be of interest to note that each of the three European mole manufacturers in the beginning selected a different kind of tool.

Wirth, starting off with tools manufactured under license to the Hughes Tool Company, concentrated on roller bits which, for use in harder rock, were of the TC button type. Some costly experiences when driving a down grade tunnel in granite in the Austrian Alps led them to a different design of the cone shape and of the bearing and sealing arrangements and they subsequently developed and have now standardized on tools of their own design. Some two years ago they began experimenting with TC studded disc cutters which proved to be more economical during a 3,500 ft. long raising job at a 60% incline for pen stock excavation in some hard Swiss granite and gneiss formations last year. Today they offer both roller bits and disc cutters as standard tools with interchangeable bearing and saddle arrangements, so that the most suitable tool may quickly be installed with changing ground conditions.

Demag have always used disc cutters, usually with two or three discs combined in one bit body. No doubt, our next speaker will go deeper into this.

The machines now marketed by Atlas Copco, have always used TC tips. As they, through their cutting geometry, differ substantially from all other types of tunnelling machines, I will now highlight some of their features.

SPECIAL FEATURES OF TUNNELLING MACHINES WORKING ON THE UNDERCUTTING PRINCIPLE

The inventor of the system for cutting rock in a radial direction by means of a number of separately driven cutter heads equipped with TC tipped tools, mounted on a slowly revolving boring head of tunnel diameter, was the late Joseph Wholmeyer, an Austrian engineer, who took out the first patent as early as 1951, two years, incidentally, before Robbins started in the tunnelling machine business. The system was, at one stage, applied by the German firm of Krupp on an experimental machine for rather soft rock formations - mudstones of approximately 3,000 psi compressive strength - and was in accordance with the original intentions of the inventor, but after his death in 1964, developed further to make it suitable for the cutting of hard rock, first by the Swiss firm of Habegger and, since the close of 1968, by Atlas Copco which firm now owns all the patent, manufacturing and selling rights.

The reason why it took some 15 years to reach the stage where efficient and reliable machines could be built according to this principle, is that it took a long time before the secret was found of how to achieve acceptable tool economy - namely, that cutting had to be carried out at low speeds (20 - 50 ft./min.), with considerable cutting depth (3/8 in. to 3/4 in./tip) and without tool - chatter, thus requiring a very rigid machine design - and that it took years of research and testing to develop tough and yet wear resistant grades of tungsten-carbide.

By inclining the cutter heads to the machine axis and by advancing the boring head which carries the cutter heads with a speed related exactly to its rotation, the cutters are caused to penetrate the rock in concentric, helical paths, cutting the walls of the tunnel like a multiple-start internal thread. (Fig. 8)

This layout makes it possible to "undercut" the rock so that only about one-third of the total volume is worked by the cutter tips, while the remaining two thirds, in the form of an uncut ridge immediately behind the cut, is broken away by a slight rearward pressure exerted by a wedge-shaped protrusion behind each cutter tip and rotating with it. (Fig. 9) Along the tunnel walls the rough surface, produced when the ridge is broken off, is trimmed by finishing cutters to protrude not more than 1/8 in. to 1/4 in. above the bottom of the groove cut by the main tools.

The radial cutting action requires little thrust against the face and the forward reaction on the tools when they undercut the rock reduces it still more. It has been found that undercutting machines need no more than about 10 to 30 per cent of the thrust required by machines working the rock with rotary tools. This is a great advantage not only because it reduces the load on the main bearing, but also because it decreases the problem of finding sufficient anchorage for the propulsion unit in soft or broken ground. In addition it simplifies maintaining alignment of the machine in rock of varying hardness. The higher the thrust, the greater is the tendency for a boring machine to veer towards the softer zones. This throws excessively high side loads on the main bearing and explains why an American-made tunnelling machine had to be withdrawn with a collapsed main bearing after boring somewhat less than 600 ft., while an undercutting machine drove a further 2,900 ft. in the same tunnel and in the same ground without any such difficulty and without ever being more than 1 1/2 in. off line. (Julia hydro electric power station, 1967/68)

It must be self-evident that far better tool economy is obtained if only a third of the total rock volume is cut than when the entire rock mass excavated from the tunnel is worked by the cutting tools.

The undercutting principle also reduces the production of fines which, especially in wet tunnels, lead to considerable wear on the muck removal system through the formation of, often highly abrasive slurries. Screen-analyses of the muck produced by undercutting machines in various types of rock show that the fraction below 3/8 in. constitutes less than 10 per cent of the total volume.

Tunnelling machines in which rotary cutters are employed work the rock face frontally so that the tools cannot penetrate sideways. Steering can, therefore, be carried out only by swinging the rear of the machine around a point close to the tunnel face. Due to the overall length of the machine, the minimum curve-radius is usually not less than 300 to 400 ft.

Undercutting machines work the rock in a radial direction their tools can penetrate the tunnel walls and steering may, therefore be executed around a point at an appreciable distance behind the face, leading to a shorter curve-radius, especially when the machines are built in articulated sections.

An interesting example in this respect is the machine which Atlas Copco started up recently at the White Pine Copper mine in Michigan (Fig. 10).

It can negotiate curves with a radius of no more than 40 ft. to the centre line and slopes of 20% up or down. The four rotating cutterheads of 4 ft. diameter are mounted in groups of two, undercutting the rock in a sweeping motion, (Fig. 11) like windscreen wipers and, like them, overlapping in the centre to produce a mainly rectangular opening, with an absolutely flat footwall and back and slightly curved sides, of 16 ft. width and 8 1/2 ft. height. These features make the machine extremely suitable for mining purposes. (Fig. 12)

The machine has been laid out to bore 1,000 ft./month, double shifting, in the slightly metamorphic, bedded White Pine ore formations of shales and interbedded sandstones with a compressive strength between 18,000 and 28,000 psi. Though it is obviously too early to say, preliminary reports indicate that this target is likely to be reached from a cutting performance point of view. Due to a series of up-throw faults, the machine operated with the two lower cutter heads working in very much harder than normal sandstone underlying the ore formations when emerging from the erection chamber. Though this sandstone obviously caused increased tool wear, the cutting speed did not suffer appreciably.

Another machine for non-circular openings now being built by Atlas Copco should be of interest to the construction industry for the excavation of the outer-most branches of sewage and water reticulation systems, cable ducts etc. under densely populated areas. Weighing no more than a total of 25 tons and consisting of two main parts of 18 and 15 ft. length respectively which can easily be lowered to tunnel level through a shaft from street level, the machine will cut openings of 4'3" width and 7'0" height in the centre, with straight, vertical sides, a slightly dished invert and a semi-circular roof line.

EXPERIENCE OF UNDERCUTTING MACHINES

So far, a total of four machines for circular openings working on the undercutting principle have been produced and put to work, apart from the original Wohlmeyer prototype.

Three of the machines, ranging from 12'0" to 13'3" in diameter and partly manufactured under license in Japan, were used for exploratory work - pilot tunnels - for the Seikan project in that country, where a railway tunnel is to be driven to link the main islands of Honshu and Hokaido. Two of these are at the moment in service and have driven some 5,000 and 2,000 ft. respectively.

These figures may not seem impressive, but they have been reached under partly very severe, adverse conditions. Large inflows of water and extremely bad ground conditions have, for months at a time, limited advance to not more than 50 ft./month. I think you will agree that it would be rather pointless to quote any figures from such a non-representative job. Let is suffice to say that excavation by conventional methods would have been completely out of the question under such conditions.

The fourth machine has been in service in Switzerland since 1967 and is now on its second job, excavating an 11'3" diameter sewage tunnel with a total length of 3 miles near the town of Rorschach.

The rock there consists mainly of a rather tough sandstone, with compressive strengths ranging from approximately 17,000 to just over 28,000 psi and containing some 60% of free quartz with a calcitic binder. For the 9,000 ft. excavated to date the contractor estimated that the average compressive strength has been in the vicinity of 22,000 psi. During this period the average advance has been 40 ft./day of 20 working hours. The best day gave 73 ft., the best 5-day week 250 ft. and the best month 1,050 ft. so far.

Average boring time of the total available working time has been just over 60% - with some weeks running as high as 75 to 80% - while tool changes and machine maintenance each accounted for 10% of the working time. The remaining 20% was lost due to delays behind the machine. I think these figures tie in pretty well with the average US results reached on a well-organized construction job.

Tool costs, including costs for renovation and depreciation of the tool holders and for labour during tool changes, have so far averaged $10.50 per running foot of tunnel. This amounts to $2.65/cu. yard excavated. For this kind of rock this must be considered as a very low figure and is, of course, due to the fact that the tools are undercutting and only work 1/3 of the total rock volume excavated.

From the engineer who runs the job down to the old man who sweeps the floors and makes the coffee, the total crew on site exists of 16 men, 10 on day shift and 6 on night shift. Total cost of the operation, including transport of the rock out to the tip and with machine depreciation calculated over a total distance of 4 miles, amount to $57/linear foot, though you will find it hard to get the contractor to admit this.

CONCLUSION

Judging from the fact that tunnelling machines working on the undercutting principle

-offer a free choice of the shape of the opening produced,
-can negotiate tight curves,
-operate at low thrust and anchoring forces,
-use one type of tool, independent of rock hardness,
-cut the rock with suitable fragmentation at moderate tool costs,

one is inclined to conclude that the undercutting principle enables us to design very flexible and economical to operate tunnelling machines.

Continuous research to improve tool life in really hard rocks, carried out in close cooperation with tungsten-carbide experts and other metallurgists, contributes towards making these machines capable of meeting the widely varying tunnelling requirements of the mining as well as of the construction industry.

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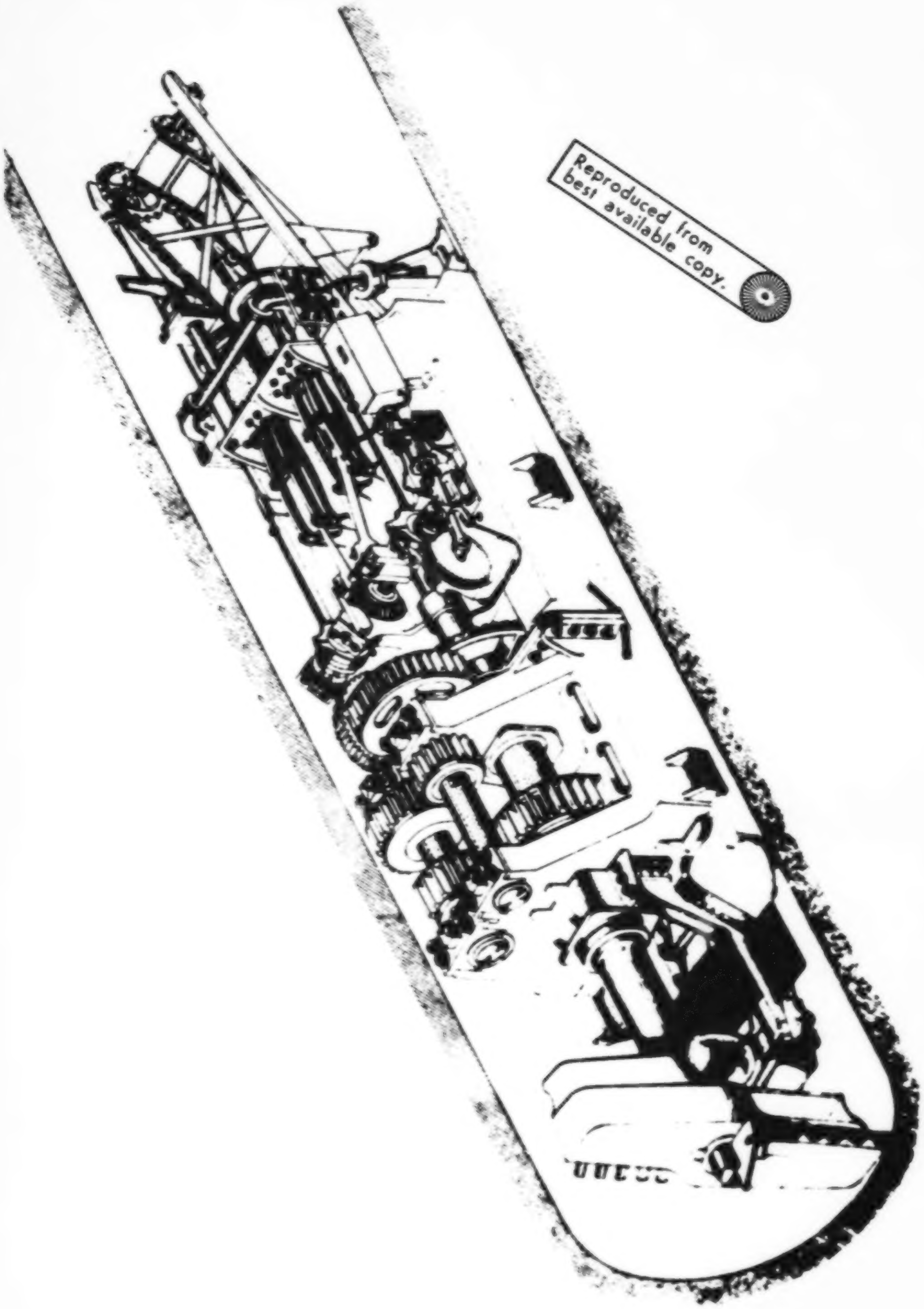
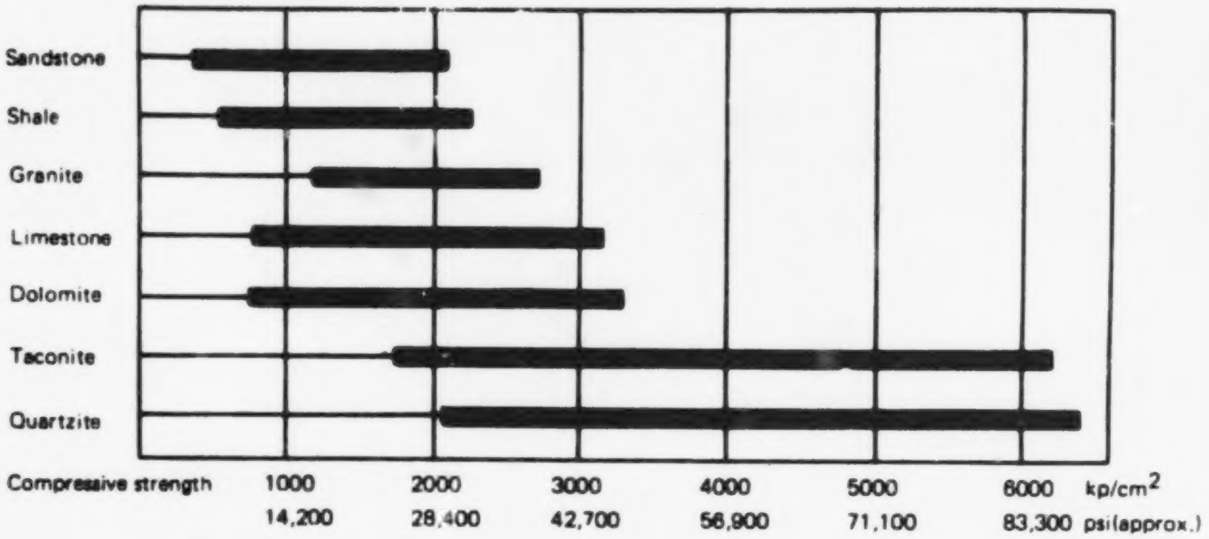


Fig.1



Variations in compressive strength of some rocks

Fig. 2

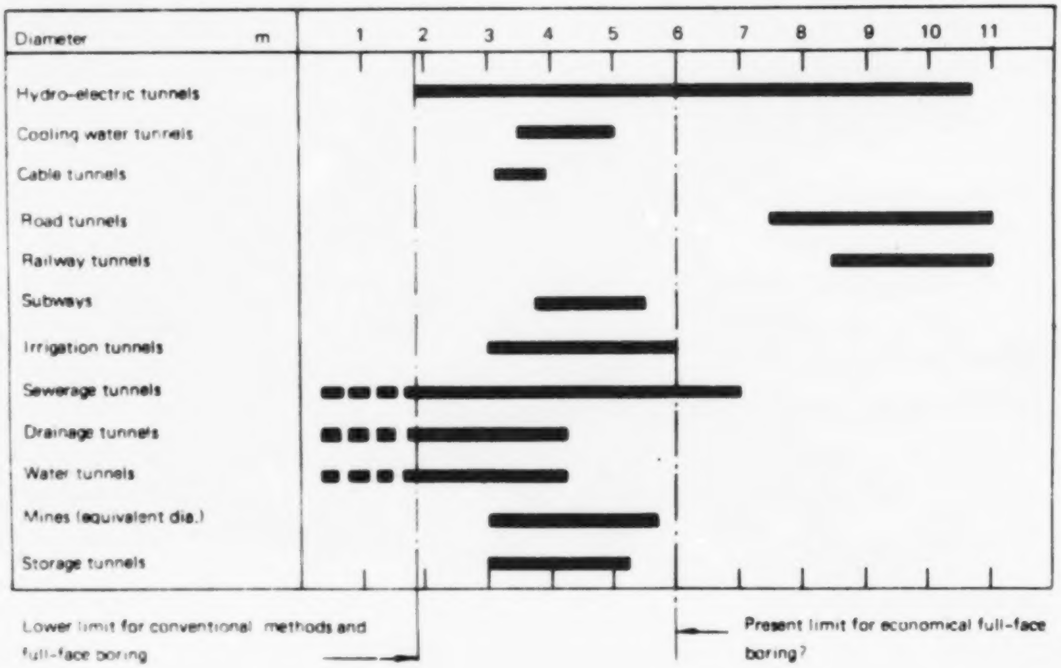


Fig. 3

Tunnel diameters and purposes

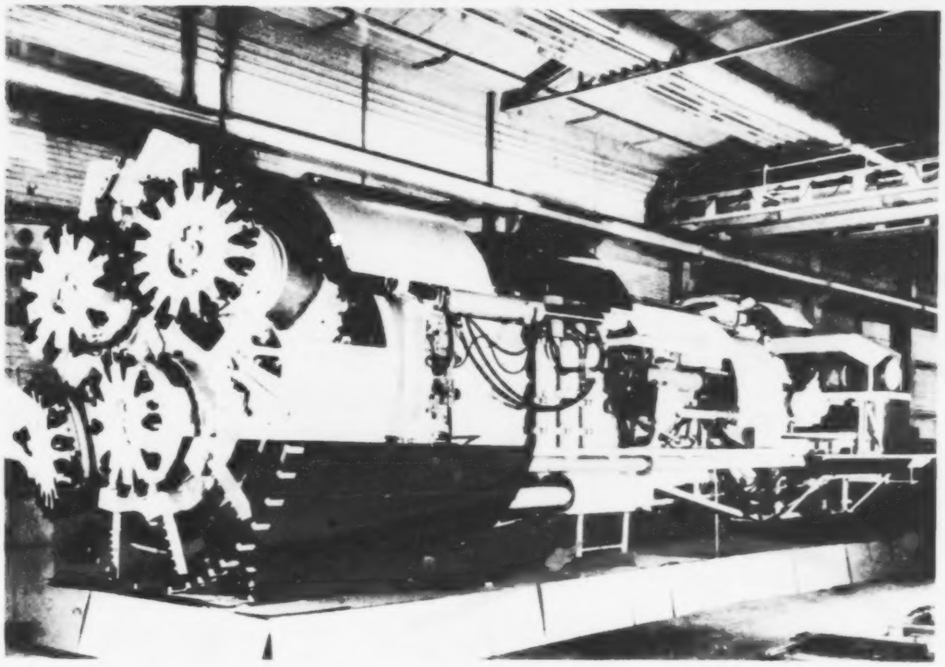


Fig. 4

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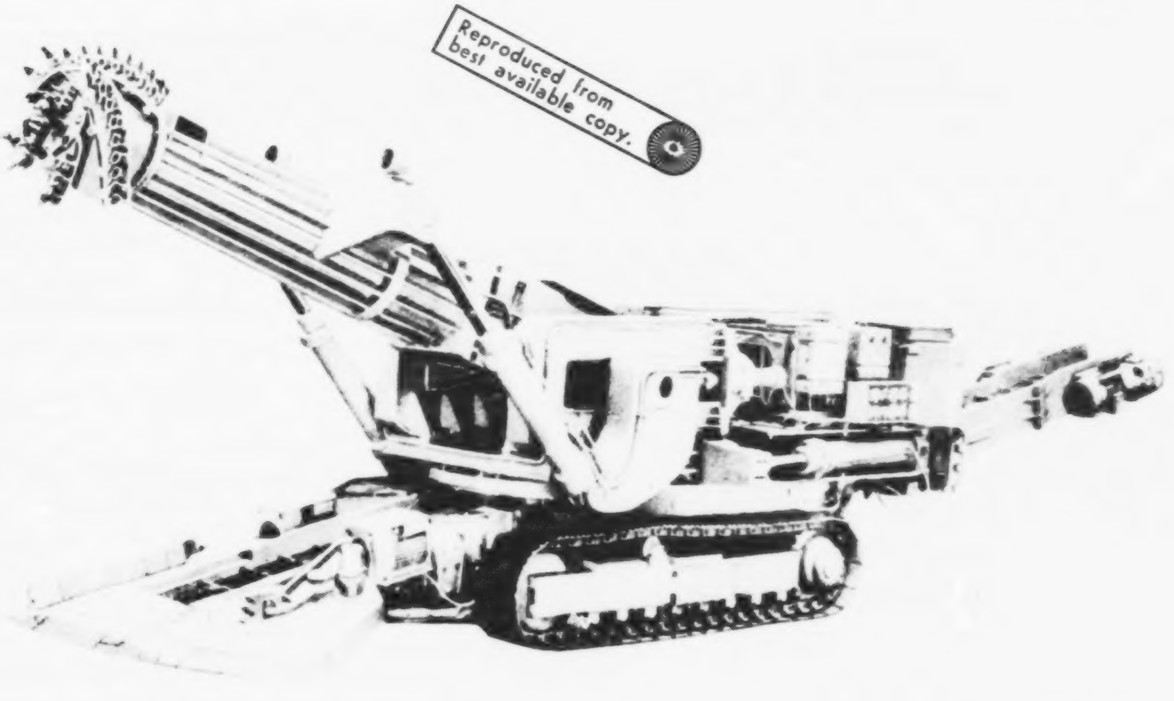

Fig. 5

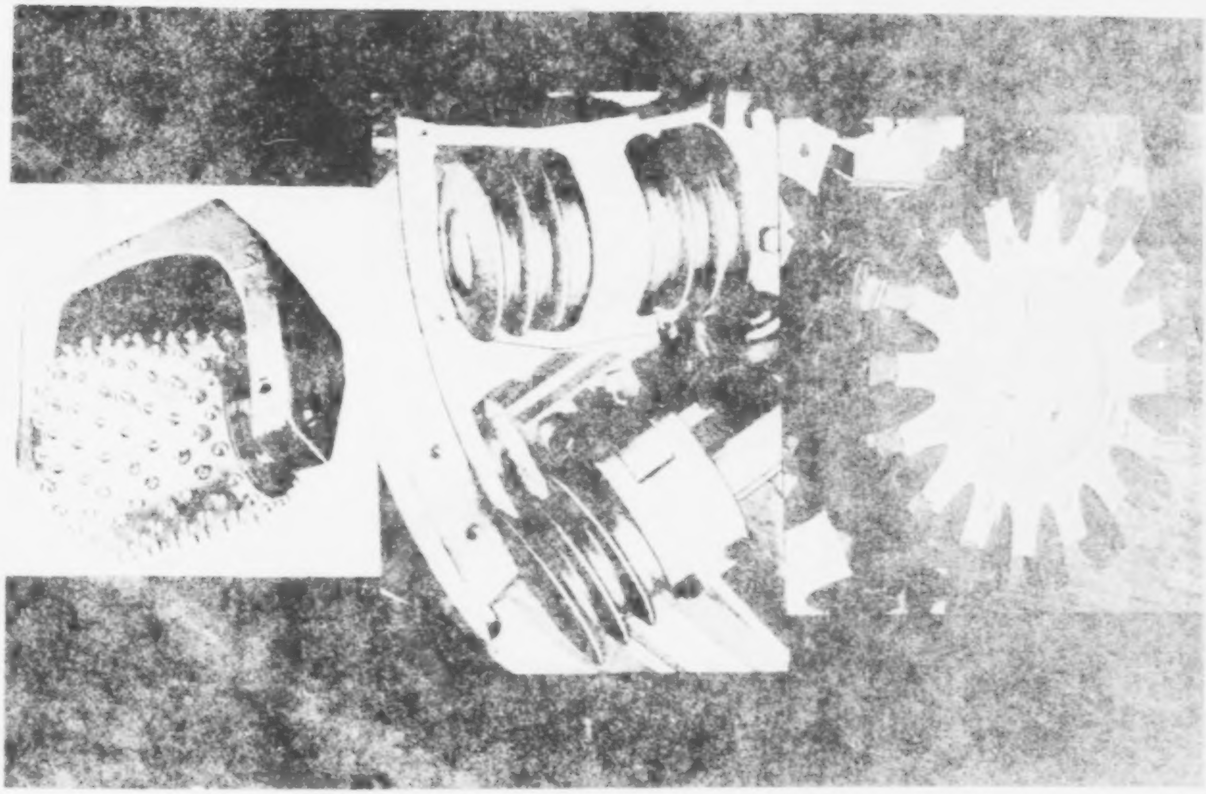

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Fig. 6abc

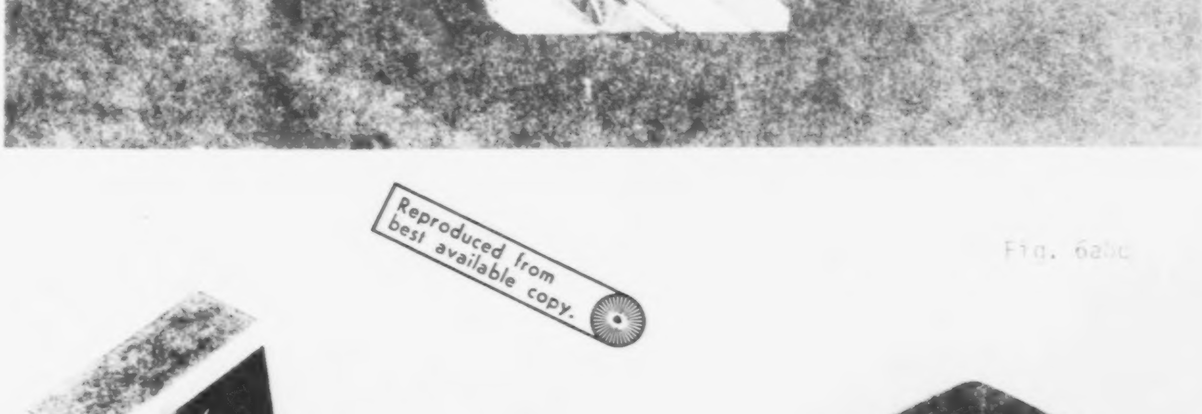


Fig. 7

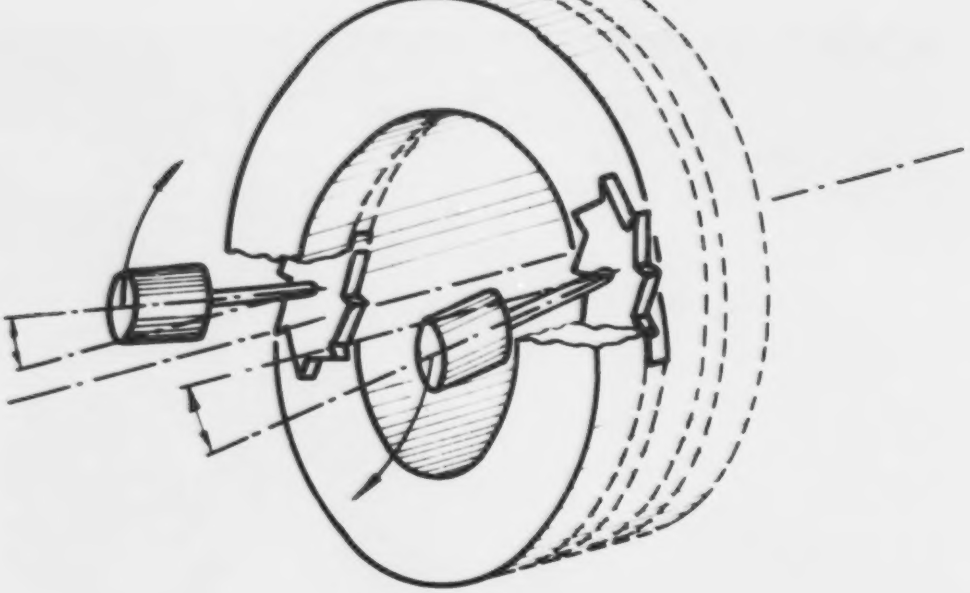
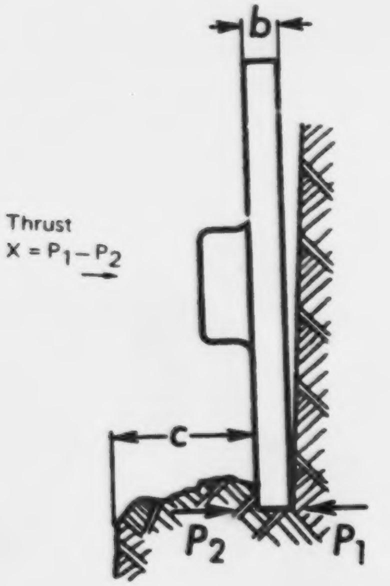


Fig. 8

b = cutting width


b
Thrust
X = P1 − P2
c
P2
P1

Undercutting towards a free surface

Fig. 9

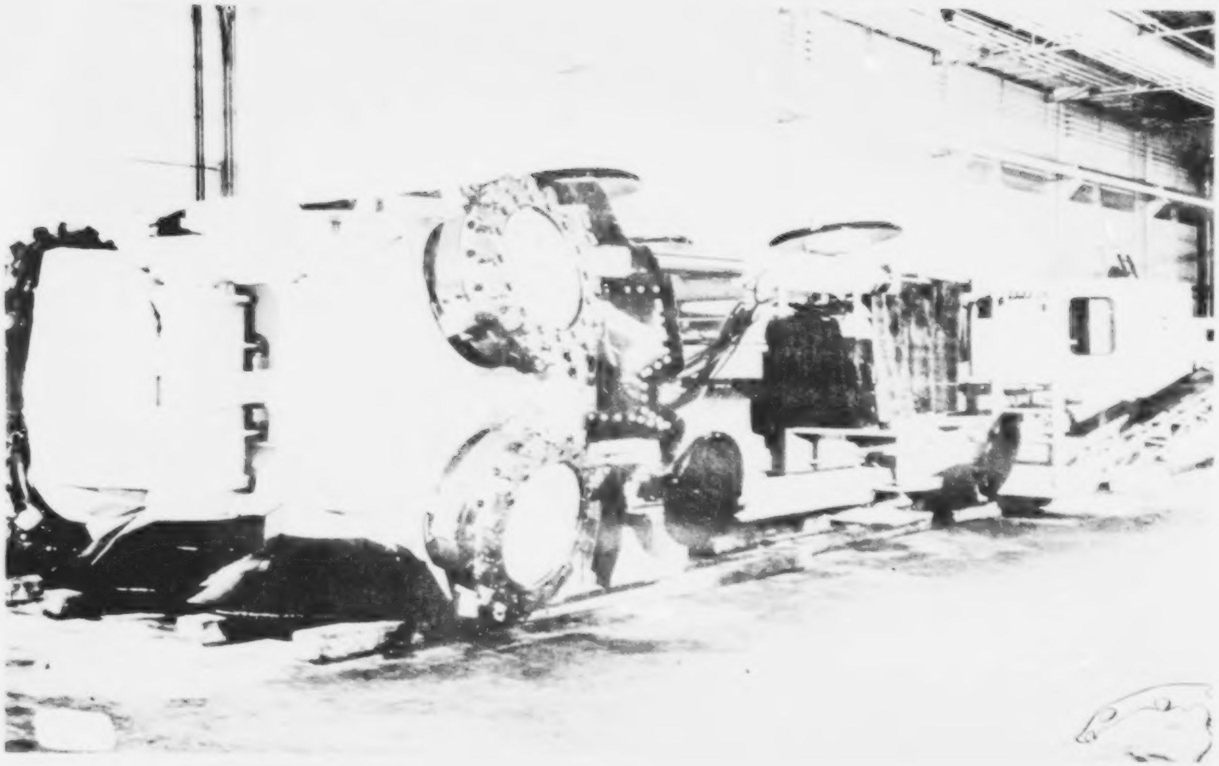


Fig. 10

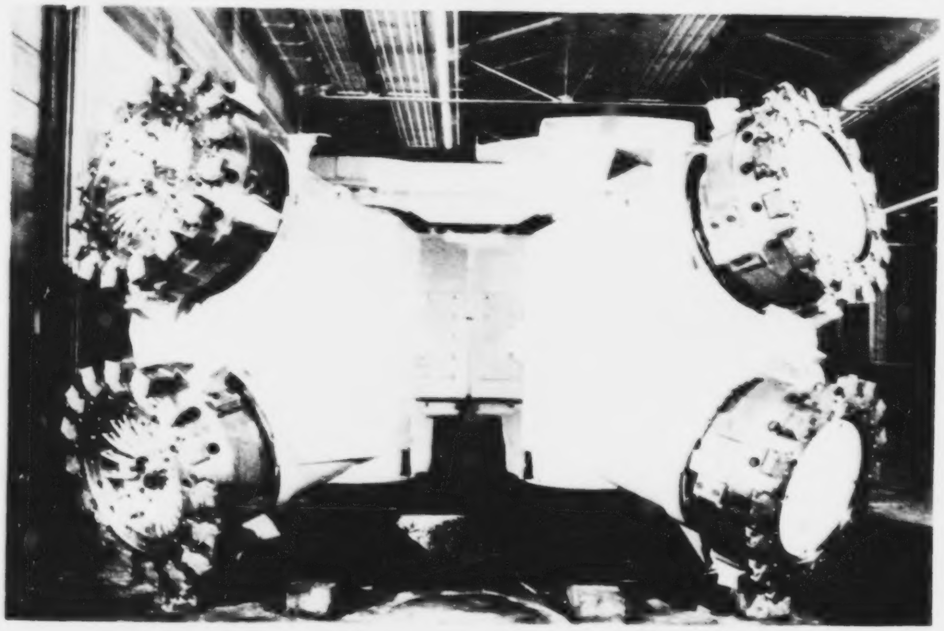


Fig. 11

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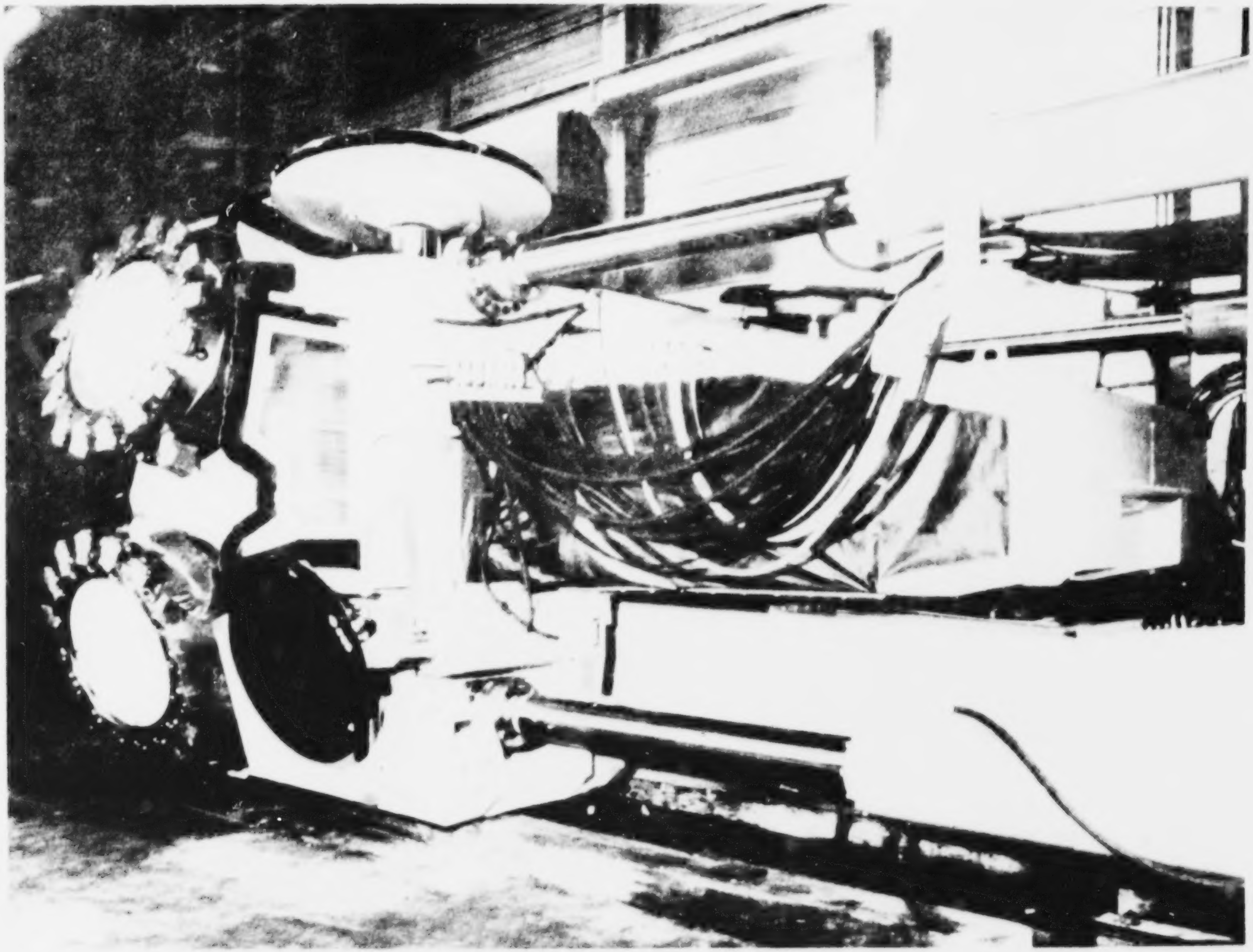

Fig. 12

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Section 7

EUROPEAN DEVELOPMENT AND EXPERIENCE WITH MECHANICAL MOLES IN HARD ROCK TUNNELING

by

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PRACTICAL EXPERIENCE IN THE FULLY-MECHANIZED DRIVING OF GATES, DRIFTS, ROADS AND TUNNELS

GENERAL

At the beginning of my paper I should like to illustrate the extent of the distances that have to be driven through rock throughout the world. On the basis of this data one can recognize the importance of fully-mechanizing the driving of gates, drifts, roads and tunnels.

At the present time the following distances are driven by the West German hard coal mining industry for an output of some 100 million tons per year: Approximately 76 mi. of stone drifts and cross cuts per year, approximately 340 mi. of gates, drifts and roads per year. This corresponds to a specific driven length of about 4 mi. per million tons per year.

An estimate of underground mining of all minerals throughout the world puts the output at not less than 1.9 thousand million tons per year. The total length of the distances to be driven throughout the world underground would hence be approximately 1900 x 4.0 mi. = 7,600 mi. per annum. The civil engineering industry (canalisation, water tunnels and similar projects) would increase this by about 5%.

It is interesting to compare these figures with statistics obtained from the South African Mining Industry. During the last 10 years in that country alone, an average of 630 mi. per year were driven underground. It is safe to assume that the remainder of the world's underground mining industry needs to drive more than 12 times the distance required by the South African Mining Industry.

The approximate 7,600 mi. per year are driven at a cost of 500 German Marks for every 3 ft. 3 in. driven ($44/ft.), equivalent to a total expenditure of at least 6 thousand million German Marks per annum ($1.7 billion/yr). With an average figure of 1 ft. per man and shift, probably more than 250,000 men are employed all over the world on underground driving operations (this takes into consideration absenteeism, etc.).

It is obvious that these figures illustrating the development of underground tunneling and mining have repeatedly encouraged the investigation of the possibility of mechanizing tunneling and drifting. A consideration of social aspects is also important. The number of accidents occurring during conventional tunneling and drifting is radically reduced by the employment of fully-mechanized tunneling equipment. Maintenance of the health and the ability to work of several thousand skilled workers underground is to be welcomed from all points of view. If it is borne in mind that underground mining is taking place at greater and greater depths and higher rock formation temperatures are hence encountered, it may be seen that mechanization also makes the miner's work easier.

DEVELOPMENT

As early as 1856 a tunneling machine was used for the first time for a preliminary investigation of the Channel Tunnel Project. Today, after more than 100 years of development works, tunneling machines employ almost the same principle.

During the last 10 years, mechanical tunneling techniques have become so sophisticated and improved that the economical employment of modern machinery is beginning to gain ground as compared with conventional methods. Drifts and tunnels, many miles long, have already been mechanically driven today, and for most projects, in addition to conventional driving, tenders are also invited for fully-mechanized driving. It is to be anticipated, that in the near future, developments will further shift the economic aspects still more in favor of fully-mechanized driving. This trend may be illustrated by a few examples:

> The tunnel in the Swabian Jura in South Germany for supplying water to the Stuttgart area from Lake Constance had a length of more than 11.6 mi., diameters of 8 ft. 2 in. and 9 ft. 2 in. and was completely driven in barely three years. Several tunneling machines were employed simultaneously at different points.
>
> Machines with very large diameters have been employed in the construction of underground railways in large cities throughout the world such as New York, Budapest, Vienna, Paris, Munich, Hamburg, Moscow and Leningrad. Partial mechanization in the construction of the underground railway in Prague, Czechoslovakia, has already been employed and is being extended.
>
> In the Harz Mountains in the Federal Republic of Germany a water tunnel about 5 mi. long is presently being driven and will be completed shortly. Very difficult conditions, such as hard rock with faults and very high rates of intrusion of water, were coped with here.
>
> Inclined shafts for power stations were drilled in granite in the Swiss Alps.
>
> In Japan it is planned to drive hundreds of kilometers of express tramway tunnels of very large diameter for linking one island to another.
>
> Beneath various large German towns, sewage tunnels are regularly driven by tunneling machines. Examples are in Dortmund

and Wuppertal.

In Liverpool a large road tunnel with a diameter of more than 10 meters (32 1/2 ft.) was built in the dock area by a tunneling machine.

Underground caverns are made by full face headers on face heading machines in hard and soft rock formations.

There is a very wide range of applications in the hard coal mining industry in Germany. Large tunneling machines are ready to start work at very great depths in difficult ventilation conditions with emission of methane.

PRACTICAL EXPERIENCE

Past Performances

Figure 1 illustrates some of the performances that have been attained to date. We have heard of conventional operation in stone drifts and cross cuts in Czechoslovakia with world record driving rates of 3170 ft./month without linings and supports. Indeed, for a period of 6 months an average of 1846 ft./month were driven.

With a full face header, more than 6160 ft./month have been attained in the USA in medium-hard rock under favorable geological conditions.

In the Harz Mountains in the so-called Kahleberg sandstone alone with a compressive strength of about 33,000 psi and containing about 65% abrasive constituents, more than 10,200 ft. has been driven to date. Rates of 890 ft./month have been attained. To our knowledge, this is the first tunnel in the world which is being driven under such conditions and in such hard rock with a tunneling machine.

In Stockholm, 394 ft./month have been attained in granite and gniess with a compressive strength of about 42,000 psi. This was in almost exclusively a single-shift operation. The extremely hard rock encountered here initiated the development of improved drilling tools.

Tunneling machines have also been developed for loosening and cutting of soft rock. A face heading machine with a soft rock cutting jib, which works in a totally-enclosed shield, was developed for driving a sewerage tunnel in England with a length of about 5 mi.

In the inclined shafts in the Swiss Alps, already mentioned, driving rates of up to 410 ft./month were attained with angles between 30 and 40 degrees. The working conditions, which on a conventional basis had been difficult, were coped with much more safely by the fully-mechanized driving system.

There is now a large number of manufacturers in the world who have devoted themselves to fully-mechanized tunneling and driving techniques. I should like to refer to the courage and the development work which mechanical engineers, contractors and project engineers have applied to problems of fully-mechanizing the tunneling and heading process. To our knowledge, in 1969, 25 mi. were already mechanically driven in hard rock alone. It is not intended at this juncture to give a detailed list and mention the whole range of types of machine. The relevant literature and papers in this specialized area have given sufficient information on this point. I should merely like to point out that all modern driving processes are based on the principle of operation with roller cutters, Kerving tools, spirals or cutter bars.

The drilling process is influenced to a decisive degree by the choice of a suitable tool. The best roller cutter tools have been developed as a combination of discs of steels highly resistant to wear. Such discs are combined with cemented carbide plates.

The roller-type tools should produce drillings which are as coarse as possible and produce optimum advance. However, since the type of rock changes frequently in practical operations, a compromise must be aimed at in the selection of tools. A special method of arranging the roller cutters on the boring head makes it possible to select different discharge widths for the conditions encountered at any time. It is aimed at developing tools in which the service lives of the bearings and the elements splitting rock are equally long. The idle times necessary for exchange are thus substantially reduced. It must be possible to rapidly remove the roller cutters towards the front by means of transport equipment. Replacement of a roller cutter proper takes only a few minutes.

Figure 2 gives a review of the service life of roller cutter tools as can be anticipated, to my knowledge, at the present time.

The rate of advance of a machine equipped with roller cutter tools is a function of the contact pressure on a tool or a function of the pressure in the hydraulic system. Every type of rock has its own characteristic phase. When a minimum contact pressure is exceeded, the rate of advance increases much more quickly. In one case we were able to establish that between compressive strengths of 36,000 and 42,000 psi, the rate of advance increased by a marked degree from 1 ft. 9 in. to 2 ft. 7 in./hr. when the pressure was increased from 220 to 280 atmospheres. At this point it is necessary to refer to the wide variety of factors which, apart from the compressive strength of the rock, also effect the rate of advance. Such factors include the shear strength, the degree of intergrowth, the particle size and the proportion of abrasive constituents such as quartz, felspar and others.

At the present time, on the basis of experience gained regarding the service life and the cost of tools, we are attempting to render possible predictions regarding the costs of future projects. We are aiming at being able to make such predictions with the aid of regression analysis from rock data corrections by EDP programming.

A problem particularly affecting the miner is the placing of linings and supports at the right time. The circular profile represents the most favorable statical form of cross section. In the event of falls or formations exerting pressure, it is important to quickly place temporary supports in the form of sprayed concrete, rock bolts, steel linings or liner plates.

Figures 3a, 3b and 3c show an example of a modern tunneling machine which permits the various types of lining and support to be fitted directly behind the cutting head or behind the front clamping unit. When a closed ring lining is employed, different types such as bell sections, "tub" sections, H-sections, liner plates, steel fabricated mats, W. Hernold System plates, etc. may be employed.

Problems of a special nature arise during driving through zones of geological disturbance. When soft or slippery zones occure following hard rock zones, this may mean that driving operations have to be stopped (when the zones are of some length) because the clamping units do not find adequate support. In such circumstances it will be necessary to carry out manual driving in front of the boring head. A conveyor arranged beneath the machine can be drawn through the head for removal of the spoil. After linings and supports have been placed the machine can then pass through the zone of distrubance under its own power.

We have already also driven into tectonically disturbed zones in which the roof was falling in front of or above the boring head. In this case the disturbed zone was filled with a quick-curing concrete injected into the rock (Figure 4). After the concrete had set the machine could be driven through these zones. In some instances this procedure had to be repeated several times.

When tunnels and other underground gates are being driven, considerable intrustion of water must be expected from the rock. At the moment, on a well-known construction site, 38 - 50 USGPM of water are being encountered. A large proportion of this occurs in the area of the tunneling machine. It has been shown that the mechanical and electrical engineering of the machine is so reliable that the purely mechanical functions are not impaired. However, this considerable flow of water causes a substantial quantity of sludge to be formed. Difficulties were encountered in removing this sludge. After intensive investigation, this was taken care of by supplementing the conveyor in the lower area of the tunneling machine by a special rubber open-top conveyor which takes care of transporting the sludge upwards to the level of the transfer point to the mine cars. This enabled the rate of advance under these unusually difficult conditions to be doubled. It may be pointed out that all other tests with cyclones, sludge filters or pump systems were not successful.

Control of Line and Level and Negotiating of Curves

Control of line and level of the tunneling machine by means of a guide beam generated by a laser has proved entirely successful. It should be particularly noted that the laser has to be aligned and monitored with extreme accuracy. Deviations from the set line and level are usually caused by negligent handling. The authorities responsible for safety have recently started publishing comprehensive regulations, and to our knowledge an operating license has always been granted. The object is to protect the health of men working in the vicinity of the laser. It should further be noted that the power (1 mW) of the lasers used today in mechanical tunneling practically eliminate any health hazard. Medical evidence is available for this.

Control engineers are at present working on fully-automating the operation of a tunneling machine. Semi-automatic control has already been proven in actual operation. This eliminates operator faults and hence prevents consequential damage. In addition, optimum efficiency of the tunneling machine is obtained. Though it may appear attractive for the technician to build or operate a fully-automated and remotely-controlled tunneling machine, this would be associated with the hazard of excessive liability to operating faults caused by the specific mining difficulties resulting from high air humidity, poor heat transfer in confined spaces. Also accessibility would cause difficulties in monitoring such a control system.

At the present time, when negotiating time, the laser has to be relocated at short intervals far too frequently. This causes considerable loss of working time and hence adversely affects utilization of the tunneling machine. A curve control system should, therefore, be developed which automatically sets the desired curve radius.

In our experience, negotiation of tight curves is associated with correspondingly higher tool costs for the outer diameter cutters. It is, therefore, best to plan as large radii as possible when planning a route.

Experienced Gained in Removal of Material

The method of transporting material inside and behind a tunneling machine influences the rate of advance to a decisive extent. On studying the cycle of operation, it is found that considerable idle times are caused by faults within this transport system.

Figures 5a, 5b and 5c show in simple form tunneling operations with different means of material handling. One essential task of the site manager of a mechanized tunneling operation is proper organization. The causes of down-times are to be established and rectified at once by means of complete and extensive supervision of operation. Down-times during operation and sometimes even consequential damage can be prevented by exchange of wearing parts during maintenance times. The degree of utilization of the tunneling machine is improved and an optimum rate of advance attained.

Climatic Problems

As a rule fully-mechanized tunneling is carried out in an area with extra ventilation. Design of the ventilation system should be very carefully planned. Air requirements depend on the number of men at the face. The volume of fresh air supplied should not be less than about 60 cu. ft. per man per minute. There should be an additional 90 cu. ft. per minute per diesel horsepower. In view of possible gas emissions and in the interest of good mixing of the air in the vicinity, the air speed should not fall below about 4 in./second. To satisfy these requirements, the diameter of the ventilation pipe should be large enough to enable the necessary volume of fresh air to be blown towards the front of the tunneling machine. Today it is quite possible to effectively operate an extra ventilation system even for small diameters over a length of more than 3 mi.

The heat generated by a tunneling machine should also be taken into consideration. An example of this is a 16 ft. tunneling machine which will shortly be employed at a depth of about 3100 ft. Cooling equipment with a rating of more than 350,000 keals per hour has to be installed.

Another measure that has to be taken is the provision of a dust shield at the front of the machine. By means of water spraying and exhausting of the dusty air in conjunction with small cyclones or other wet and dry dust precipitators, the dust content in the area where the operating personnel are located is reduced to figures which exlude the possibility of silicosis and meet the official regulations. In this connection it may be mentioned that in many cases where tunnels are being driven it was proven unnecessary to have additional dust precipitating equipment in an apparatus arranged behind the tunneling machine.

Design Features of the Tunneling Machine

One of the duties of the manufacturer of the tunneling machine is to make provision for its quick assembly and dismanteling. The very cramped space conditions demand optimum design and dimensioning of the individual components so that transportation as well as repairs can be carried out without extra excavation being necessary in the tunnels and in horizontal and vertical bottlenecks.

When starting to drive tunnels directly from the surface, it has already been possible to commission an almost completely assembled machine within one week on a well-prepared site. Between 2 and 6 weeks would probably be needed, depending on the size of the machine, for delivering and erecting a machine. Conscientious examination of these transport and erection aspects must, therefore, be an essential part of any preliminary calculations.

Tunneling Machines in Soft Rock

A soft rock can be cut or kerved. The experience we have gained is limited to machines which are of the face heading type. In other words, one or more jibs fitted with cutter heads carry out selective winning at the face.

The most favorable employment conditions arise when the rock formation remains stable until the lining has been placed behind the machine. Under such conditions maximum rates of advance can be obtained. This, of course, is subject to the availability of a well functioning organization such as I mentioned earlier in the case of the machine for tunneling in hard rock. Face heading machines mounted on crawler tracks are employed for such conditions. However, their employment is limited by the angle of inclination of a section in the longitudinal or transverse directions. I, therefore, wish to talk about machines which can also cope with this difficulty. A technique is employed in which the machine is hydraulically clamped all round. With the clamping system it is possible to carry out a course correction or to negotiate curves. A great deal of experience has been gathered in this area in recent times. Course corrections can be carried out by up to 6 degrees in single steps. In undulating rock formations in particular, very frequent course corrections are necessary, since the seam has to be accurately and directly followed.

A so-called template control system has proven very good for permitting a cross section of any form to be driven accurately. This equipment permits driving operations to be carried out manually or automatically from the control stand.

In our experience the alternate clamping of the machine may lead to considerable stressing of the rock formation. A so-called "step-moving" effect occurs. This can also be observed in long wall coal mining when progressive props are employed. To counteract this, more recent machines have other types of prop heads. They enable the pressure to be reduced to very low figures. It is difficult to measure this exactly. Practical results have shown that the "step-moving" effect is largely eliminated by this means. One such machine of this type for soft rock has a flat shield which practically consists of single heads of wide area.

The rates of advance attained to date by such a machine in coal are about 33 to 50 ft. in one shift of 8 hours. When coal and soft cuttable subsidiary rock occur together, rates of advance of 25 - 30 ft. can be obtained in a shift of 8 hours. The consumption of tools for cutting loose material in the coal seam is about one pick per 3 ft. 3 in. of advance, equivelant to about 30 - 40 DM/meter ($8-11.00/m). This is subject to continuous maintenance and checking of the picks.

In the coal mining industry in West Germany, a method of mining has been tested in which longwall mining is employed with the aid of progressive props in a downward gradient of 50 degrees. This method of mining encourages coal mining engineers to anticipate economic mining of reserves of coal which are located in steep seams. This means that driving operations have to be carried out in seams with downward gradient in which the clamped face heading machines are employed. The first results are probably to be expected in 1971/1972. In the meantime, experience has been gained with operation of face-heading machines of shield like design.

Figure 6 shows the design of a machine which will shortly be delivered. It consists of shields which are connected together in telescope fashion. The front shield is used for driving, the rear one for clamping. The machine is to be employed in a rock formation with a strong tendency to expand, but which is nevertheless easily cut.

Figures 7a and 7b show a quite different version. A face heading machine mounted on an excavator is intended to enable very large cross sections to be driven, for instance, for underground railway stations. A machine such as this can drive a maximum width of 36 ft and a height of 33 ft. Simultaneous driving with several face heading jibs mounted on a special frame is also conceivable. No experience is as yet available of this.

Training of Personnel

Driving with fully-mechanized systems and the forms of organization associated with this require detailed training of skilled personnel at an early stage. Personnel should, therefore, be intensively trained for at least 4 weeks before the machine is used for the first time. Particular attention should be paid to the relevant electrical and hydraulic systems as well as to the problems involving surveying.

QUESTIONS OF ECONOMICS

Only a few aspects of this extensive and important area are touched upon here:

Capital Investment

Figure 8 is intended to give a comparison of capital investment costs for equipment for driving tunnels with a cross section of about 175 sq. ft.

Costs

Driving costs are roughly sub-divided as shown in Figure 9. This shows that fixed costs always have to be assumed for the site equipment, etc. The variable costs display a logical trend in their relationship, labor, tools, power, etc. With high rates of advance, the tool costs rise and the cost of power, capital and labor fall. Moreover, in any calculation, the indirect costs or savings derived from methods of support resulting from the fully-mechanized method of driving should not be disregarded. For instance, the circular, smooth excavation permits the use of a substantially lighter steel or thinner concrete lining. On one site in West Germany, for example, the economics of the operation were very positively influenced as compared with the older method since, on completion of driving, the lining with steel pipe and concrete backing was placed much more quickly than had been predicted.

Some specific data may serve to illustrate certain types of cost:

a) The power consumption is about 15 - 20 kwh/35 cu. ft excavated.
b) The chisel costs as shown in Figure 2 vary widely depending on the type of rock: between about 5. - 100. - marks/35 cu. ft. ($1.40-$28.00/35 cu. ft.).
c) Depreciation and interest depend on the life. It may be theoretically assumed that a tunneling machine has a longer life than 5 years. In the case of a hard-rock tunneling machine this should be equivelant to a distance of about 6 - 12 mi.
d) The cost of spare parts average about 5% of the new cost of the machine per year.
e) Lubricants and water consumption are very minor parameters.

CONCLUSION

I have attempted to give you a slight insight into practical experience in fully-mechanized tunneling. One could, of course, say a great deal more about each of the problems involved. However, I hope I have succeeded in creating the impression that the dynamic development in tunneling in recent years has led to great success and is a practical proposition.

Increasing labor costs and the striving towards a higher standard of living and shorter working week will shift the cost limit between the conventional and mechanical methods of tunneling more and more in favor of the latter.

Examples of Headings

Heading	Monthly Headway	Country
Conventional	3.435' (1.047 m)/month max. average 1.970' (600 m)/month	CSSR
fully mechanized	6.550' (2.000 m)/month rock of medium strength	U S A
fully mechanized	1.000' (300 m)/month in sandstone (32.200 psi= 2.400 kp/cm^2) (up till now more than 13.200'= 4.000m)	Germany Harz-mountains
fully mechanized	426' (130 m)/month granite (fine grain sized)-gneis 48.400 psi= 3.400 kp/cm^2)	Sveden
fully mechanized inclined shaft	440' (135 m)/month ingranite (coarse grain sized)	Switzerland

Fig. 1

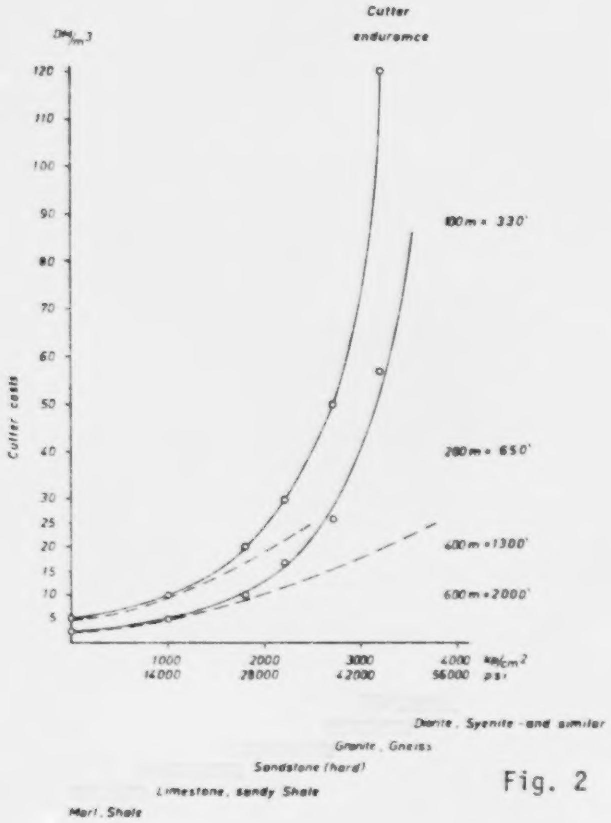

Fig. 2

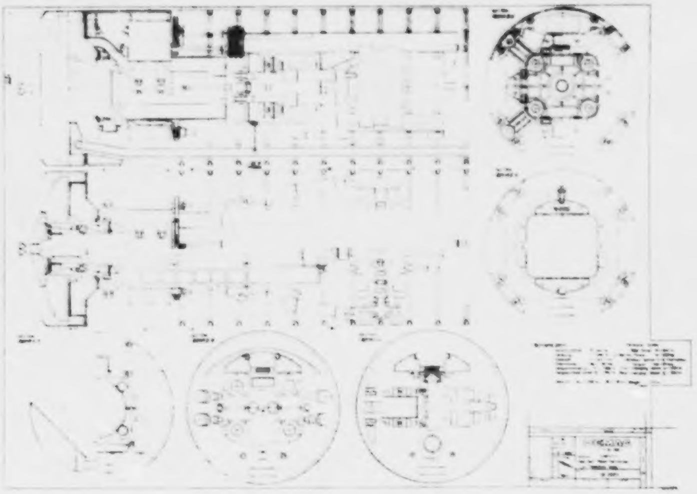

Fig. 3a

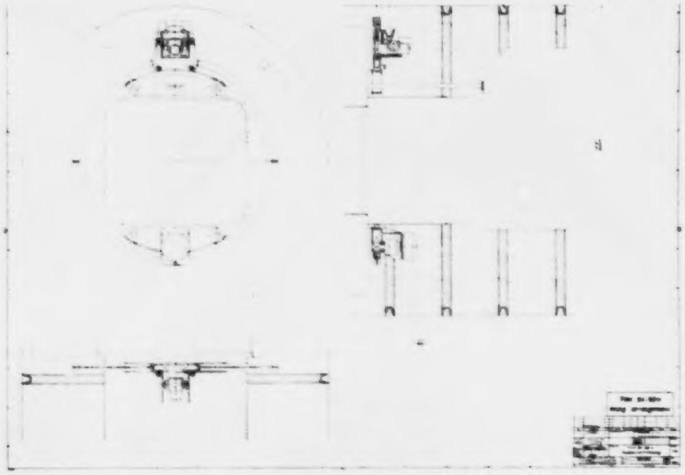

Fig. 3b

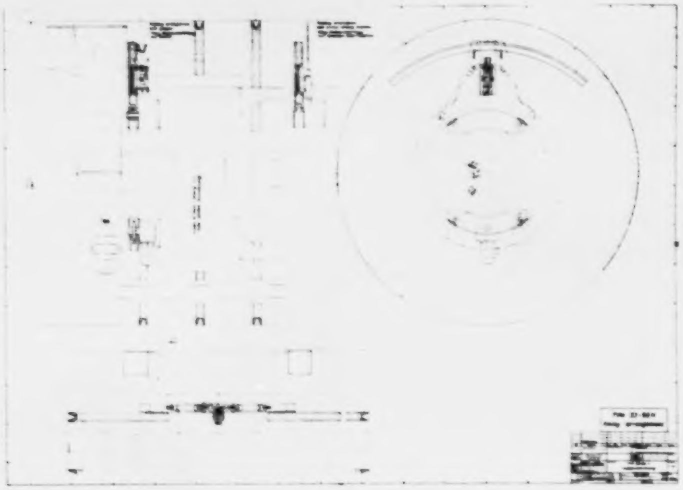

Fig. 3c

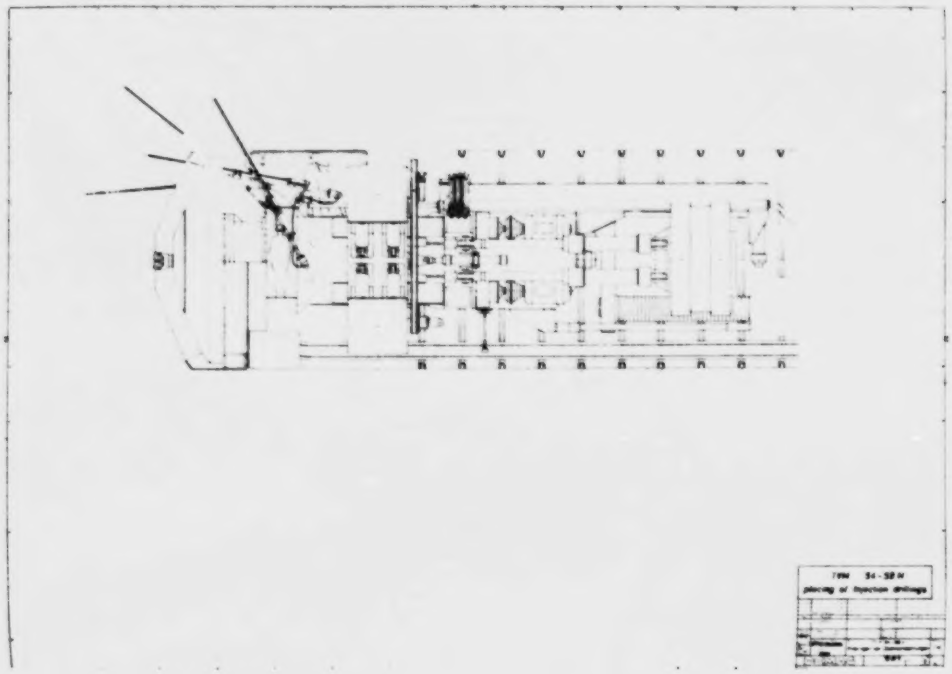
TBM 34-38 H
placing of injection drillings

Fig. 4

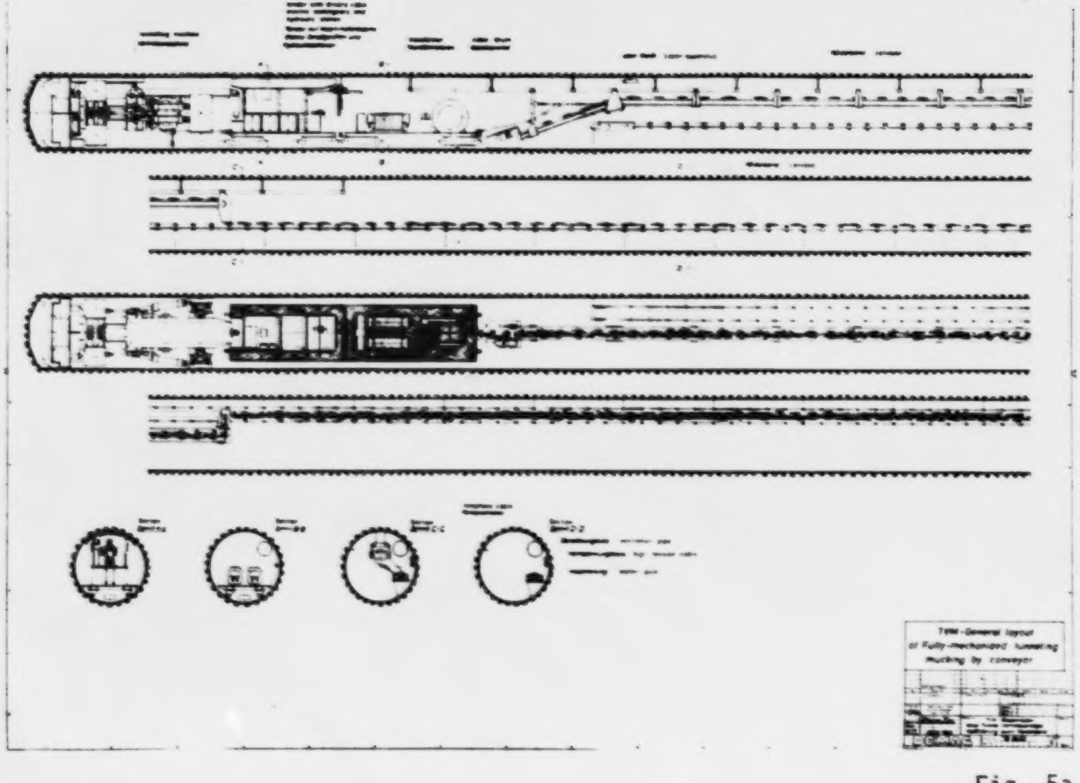
TBM-General layout
of Fully-mechanized tunneling
mucking by conveyor

Fig. 5a

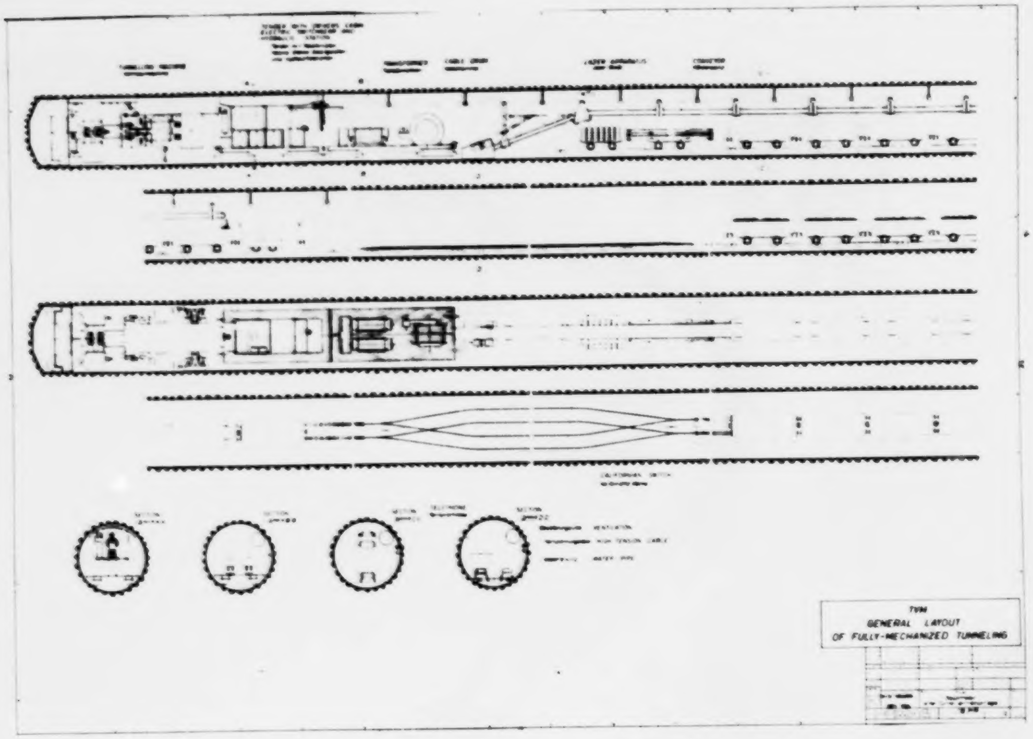
TENDER WITH DRIVERS CABIN
ELECTRIC SWITCHGEAR AND
HYDRAULIC STATION
TRANSFORMER
CABLE DRUM
LAZER APPARATUS
CALIFORNIAN SWITCH
SECTION
TELEPHONE
VENTILATION
HIGH TENSION CABLE
WATER PIPE
TVM
GENERAL LAYOUT
OF FULLY-MECHANIZED TUNNELING

Fig. 5b

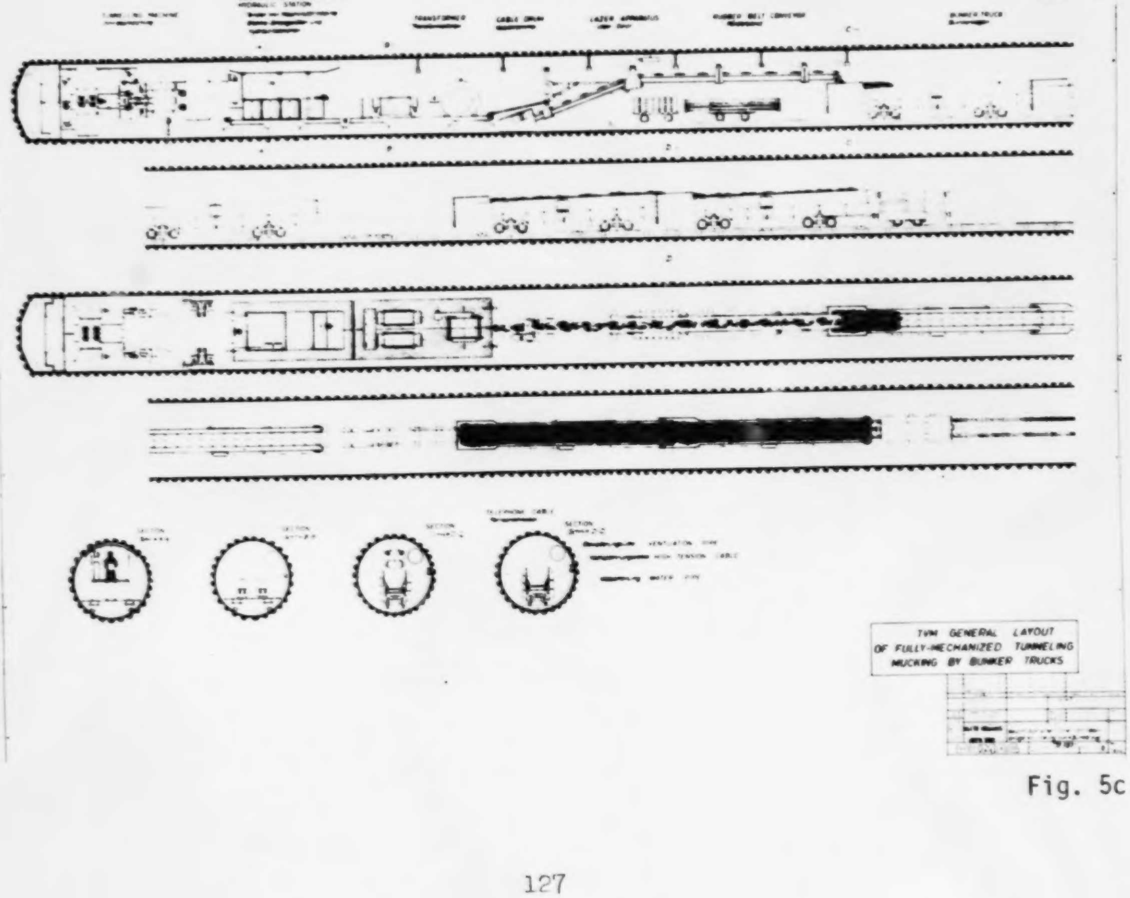
TUNNELLING MACHINE
TENDER WITH DRIVERS CABIN
ELECTRIC SWITCHGEAR AND
HYDRAULIC STATION
TRANSFORMER
CABLE DRUM
LAZER APPARATUS
RUBBER BELT CONVEYOR
BUNKER TRUCK
SECTION
TELEPHONE CABLE
VENTILATION PIPE
HIGH TENSION CABLE
WATER PIPE
TVM GENERAL LAYOUT
OF FULLY-MECHANIZED TUNNELING
MUCKING BY BUNKER TRUCKS

Fig. 5c

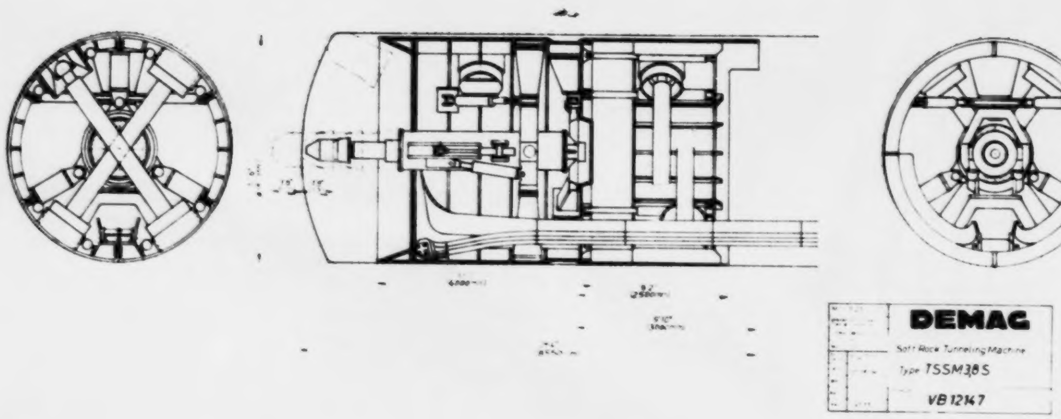
DEMAG
Soft Rock Tunneling Machine
Type TSSM3,8S
VB 12147

Fig. 6

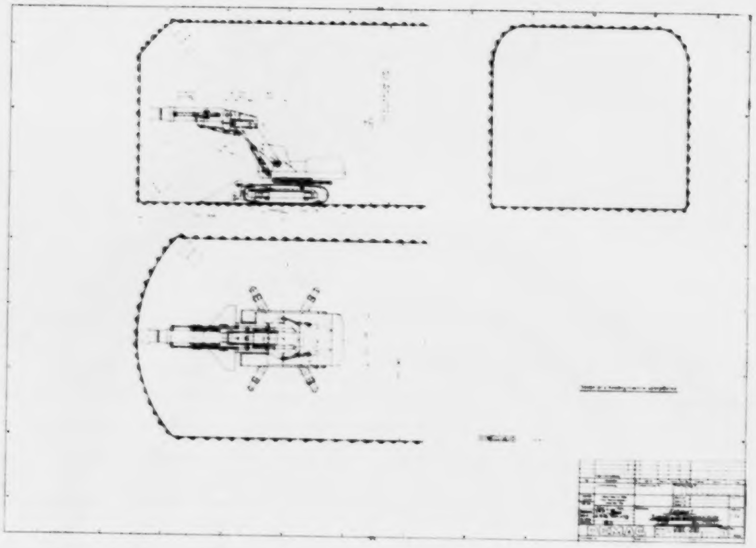

Fig. 7a

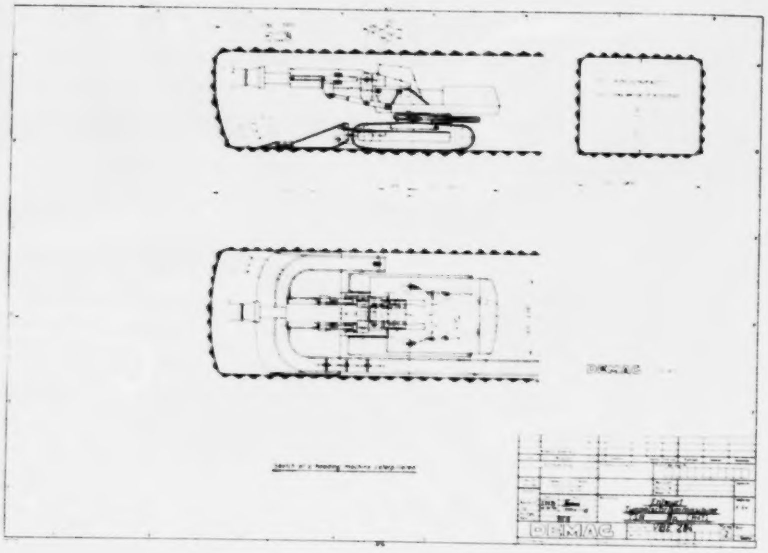

Fig. 7b

Comparison of Capital Investment Costs
to fit out Tunnel Headings

Heading	Equipment	Capital Investment Costs
Conventional	mucking and site equipment	approx. 1.7 Mio DM (490.000 US $)
fully mechanized	Tunneling machine mucking and site equipment	approx. 4.0 Mio DM (1.150.000 US $)
fully mechanized	soft rock tunneling machine with shield mucking and site equipment	approx. 2.5 Mio DM (715.000 US $)

Fig. 8

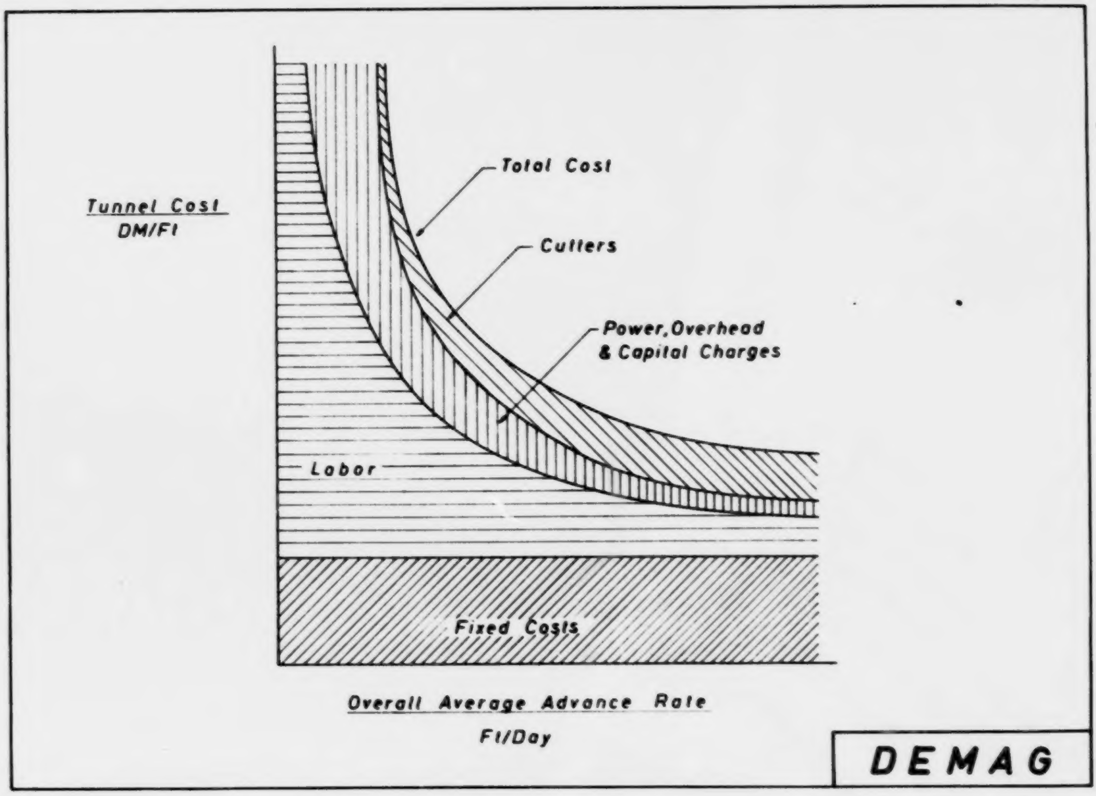

Fig. 9

Section 8

EXPERIENCE IN EDMONTON CANADA WITH EMPHASIS ON PNEUMATIC CONVEYANCE OF MUCK

by

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Mr. Chrysanthou died in December, 1970. This paper was prepared from a recording of his oral presentation without the benefit of his editorial review.

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EXPERIENCE IN EDMONTON, CANADA, WITH EMPHASIS ON PNEUMATIC CONVEYANCE OF MUCK

The City of Edmonton, the capital of the province of Alberta in Canada, is situated on both sides of the North Saskatchewan River. Edmonton proper has a population of 460,000 and a surface area of 86 sq. mi. The Saskatchewan River in Edmonton varies in width from about 400 ft. to about 600 ft. during flood conditions. The variation in level between low and high water is about 15 ft., but on the occasions of one or two extreme floods it has been 30 ft. The flow at low water is about 1200 cfs and at average high water 50,000 cfs. The greater part of the City lies on a plateau at an elevation 2200 to 2260 ft. above sea level. There is from 146 to 206 ft. above the river at low water.

The City of Edmonton is built upon surficial deposits of variable thickness underlain by Upper Cretaceous strata. The surficial deposits of Late Pleistocene age consist of a well sorted pre-glacial sands and gravels, glacial till and pro-glacial lake deposits in an ascending order. The bedrock of Edmonton is composed of upper cretaceous shales interbedded with bentonitic shales, sandstone reefs and coal seams.

During the prewar years the majority of the sewage system discharged directly into the river. Only two small treatment plants of very limited capacity were in operation. With the sudden and rapid growth of the City after the war years, it became evident that the two treatment plants were too small and could not handle the large increase in sewage flow. The concentration of raw sewage in the river got so high that the oxygen content of the river was drastically depleted. The conditions got so bad that some of the municipalities down stream were unable to use the river as a source of domestic water supply. A firm of consulting engineers was retained to study the problem, and commissioned to design a sewage system to conform with the new government regulations.

The fact that the water supply is drawn from the river at a point nearly opposite the center of the City affected the general design and made it necessary to collect all sewage below this point. The consulting engineer's recommendation was for a major central sewage treatment plant with an extensive collection system of interceptor tunnels in lieu of a series of small plants located at various locations with a series of small lateral collectors. As a result of this recommendation, tunneling for interceptor sewers was adopted as a major phase of our sewer construction program.

All the tunneling to date has been constructed by City forces. We presently have six tunneling crews working with seven tunneling machines. We average 5 1/2 mi. of tunnel construction per year in sizes varying from 4 ft. to 21 ft. in diameter with depths varying from 50 to 180 ft.

Except for small lateral columns, which are all hand dug, all our tunnelling excavation is done with tunneling machines or moles as they are commonly known in the industry. Our working shafts are located such that a minimum of 3000 ft. of tunnel is excavated from each heading; two headings are worked from each shaft. It is an accepted fact that the success of any tunneling operation is a direct function of the materials handling operation. At the last OECD Conference held in Washington, D.C., materials handling was placed as a second item in priority in the field of research and development. In tunnel sizes of 10 ft. and upwards this problem is fairly well controlled. The use of California switches and other switching devices enables us to excavate at a fairly constant rate. It is in the small diameters where the problems become critical. Fifty per cent of the tunnel excavation is in the 7 ft. diameter range. It is impossible to install a California switch behind the machine for this size of tunnel. The cutting of switching stations in hanging wall is an expensive proposition if the run is short. Consequently, the only switching of the train is done at the shaft station. These limitations cause a considerable loss of excavating time. Several time studies of the operation as a whole indicated that the mole was operational for only 43% of the time on a 10-hour shift, the balance of the time was spent in waiting for the empty trains and installing the primary liner. Consequently, a new system of material handling had to be implemented to increase the excavating time of the mole.

At the last symposium on rapid excavation held at Sacramento, Mr. Graham Ball of Radmark Engineering (head offices in Portland, Oregon) presented a paper on pneumatic conveyor systems presently used by Consolidated Mining & Smelting in Kimberly, British Columbia. The system is used to back fill cut and fill slopes. We were impressed with the system and we felt that this may be the solution we were looking for. Consequently, Radmark and the City of Edmonton agreed to cooperate to test the Radmark's stowing system as a method of removing materials discharged from a tunneling machine.

The pneumatic system was chosen because it should greatly increase the percentage of time that the tunneling machine can work. As long as a pneumatic system is designed to handle the maximum discharge from the mole at the maximum conveyance distance, then the mole should work at full capacity at all times. By blowing the material directly into a truck bin, what I call the back stop on the surface, the regular headframe and hoist can be eliminated. Access shafts can be reduced in size. The discharge pipe can be installed into 36 in. diameter holes spaced on 800 ft. centers along the route of the tunnel. The access holes are needed for the supply of electric power to the mole as well as assist in the ventilation of the tunnel. The working area in congested and residential districts is vastly reduced. The pneumatic system will also assist in the ventilation of the tunnels.

The pneumatic system was first installed in a 7 ft. diameter tunnel. Due to some difficulties encountered in the installation and operation of the system, which I intend to refer to later on in this paper, the system was removed and installed in a larger diameter tunnel. The system is presently operational behind a 12 ft. tunneling machine.

The pneumatic system consists of a large volume, low pressure, air blower installed in a closed-in trailer positioned at the surface of the shaft head. The air is piped to the stowerthrough a 12 in. diameter quick coupled pipe. The Radmark feeder or stower is connected to the mole by means of a draw bar. A hopper is located directly under the discharge conveyor from the mole. Two telescopes have been provided behind the stower, one for the air pipe and one for the materials handling pipe to permit the blower to travel forward with the mole. When the excavation has advanced 10 ft., the telescopes are fully extended and the stower is shut down. The telescopes are then retracted, a 10 ft. length of pipe is coupled into each line, and the excavation proceeds. The controls for the surface blower are located on the stower control panel underground and the over load protection will automatically shut down the pneumatic system in the event of any blockages. The materials handling pipe is a 10 in. cast iron pipe with an internal surface hardened to a hardness of 630 Brinell. An overhead monorail is continually extended to supply all the necessary construction materials to the face.

At this point Mr. Chrysanthou showed a film of the pneumatic system in operation. This prompted the following discussion.

QUESTION: What is the maximum size you are striving for there?

CHRYSANTHOU: We pass 3 1/2 in. size particles.

QUESTION: Have you used it long enough to get any idea on pipe cost or wear?

CHRYSANTHOU: No.

QUESTION: Why couldn't you drop it into a hopper and blow directly out of a hopper?

CHRYSANTHOU: Yes, we have done that too. This is basically what you have to do when you go into a residential district, you can't blow.

QUESTION: What is the total distance of the pipe from your machine to the top?

CHRYSANTHOU: About 450 ft.

QUESTION: What is the maximum that you can go?

CHRYSANTHOU: We propose to go 800 ft. laterally and 150 ft. up.

QUESTION What is the consumption of the compressed air for 100 cu.

yds?

CHRYSANTHOU: I don't think I can give you that detailed information, but the blower has a capacity of 5800 cfm at 18 psi. It is powered by a 500 hp motor.

Our comments on the system so far are that the basic idea of the stower following the mole is good. The capacity of the pneumatic system is adequate and will not restrict the boring rate of the mole. The stower is capable of handling 3 1/2 in. material and larger sizes are broken down to size by chopping. The equipment can be operated by tunnel laborers. The abrasion factor of the material is low, so pipe and elbow wear should not be a problem. Because of the dampness of the material, dust is not a problem.

In the initial installation of the pneumatic system in a 7 ft. diameter tunnel, the small diameter of the tunnel compounded with the fact that the tunnel was started on a curve caused various inconveniences to creep up. The lack of working space make it awkward and frustrating for men to work efficiently. The system provided for carrying the telescopes behind the stower was designed for straight tunnels. The telescope skid could not center itself in the tunnel, therefore, it tended to climb up the ribs and not remain under the telescope. Only one ball joint was supplied and this was located at the back of the stower. The operators had a difficult time lining up pipe joints when a section of the pipe was to be added to the telescope. The materials handling pipe was too rigid and we had difficulty in forming it to the radius of the tunnel. The air pipe could not be located over the material pipe. It, therefore, lay on a radius different than that of the material handling pipe. Joints in this pipe would get ahead of the joints in the material handling pipe and, therefore, pipes spools of different lengths were required in the air pipe.

The pneumatic system bogged down when large sizes of material were fed to it. The presence of these large pieces could not be avoided and the balance of the material bridged in the hopper. This necessitated the shutting down of the feeder and manual removal of the bridged material. A second set of choppers was installed to break down the large pieces; thought it did help some, it was soon discovered that a larger, faster and more powerful unit was required in order to keep up with the discharge rate of the mole conveyor. A build-up developed in the elbows. As this build-up increased, the operating pressure increased. The back pressure was high enough at times to activate the automatic shut-off. No cleanouts were provided for the elbows and, since the build-up was very high, a lot of time was consumed in dismounting and cleaning the elbows.

The pneumatic system handled the shales and the stiff clay but completely broke down when a pocket of quicksand was encountered. The water in the formation would mix with the clays and turn the clay into a stickey mess that would instantly clog the materials handling pipe and the elbows. It was at this stage that a decision was made to remove the unit and return to the conventional materials handling method.

You may at this point feel that I have painted a fairly dark picture of the pneumatic system. This is not the case. We did not expect any miracles from any untried system right off the start. We undertook the use of the pneumatic system as a research project. Since it is to my knowledge a first of its kind in that particular application, we felt that the condition should be presented to you exactly as it happened.

The system was reinstalled in a 12 ft. diameter tunnel. Our test holes indicated that the tunnel would be excavated in a continuous shale formation. Prior to putting the system in operation, the following modifications were made:

1. A second ball joint was added to the end of the telescopes. This enabled the materials handling pipe to better maintain its alignment.

2. On a curved portion of the tunnel, a five degree miter-bend was added at every second joint of the materials pipe.

We are now considering the use of ball joints at every second joint of the materials handling pipe. These would be of value even in straight tunnels.

A second telescope will be added to the air pipe line. This could be adjusted to keep the flanges of the main telescope side by side. The pipe installation procedure would be speeded up by changing both pipes at the same time. Mark II type elbows were installed. The elbows are fitted with clean-outs for easy access and inspection. The hopper on the stower has been enlarged and a more substantial set of choppers installed. The movie that you saw was after the innovations were put in it, except for the chopper. The choppers will be capable of handling the large pieces as fast as they are discharged from the mole conveyor, thus, drastically eliminating any chances of bridging in the hopper. We are also seriously thinking of putting in a vibrating grizzly at the top of the hopper that would feed into a small jaw crusher attached inside the hopper. This would take care of the large pieces of sandstone and hard material. The throat of the stower was enlarged to allow larger materials to pass.

The present application of the pneumatic system is working satisfactorily. Last week we had the best day of the pneumatic system in a 12 ft. tunnel. We reached 55 ft. in 20 hrs. in two 10-hour shifts. There were quite a few breakdowns and quite a few revisions to be made, but it was the best day we made so far. Nevertheless, we still consider it to be a research project with room for improvement. There is no doubt in our minds that some further changes will be made before we are completely satisfied.

In conclusion, may I say that the pneumatic conveyance system has its limitations, in the art of softwork tunneling. It should not be considered if there is any chance of encountering wet clays or quicksand in the course of the excavation. The application of the pneumatic system is limited by the diameter of the tunnel. Where switching systems and double tracks can be installed, the conventional method of materials handling is still more economical. In our opinion, the system can best be used in a hard rock formation trailing a hard rock machine. The pneumatic system is ideally suited for free flowing materials such as rock cuttings , gravel or sandstone. The nearly uniform size of the materials excavated by the hard rock machine would eliminate any feeding problems. Though an abrasing problem may develop, the material should be more predictable and easier to adapt to.

QUESTION: What about the noise on the surface from the blower?

CHRYSANTHOU: The blower is installed in a semi-trailer, a closed-in unit, and the walls of the trailer are insulated with 4 in. fiber-glass bat insulation. Really the noise problem is in the range of 75 decibels which is still within 15 decibels below our noise law.

QUESTION: What about dust at the chopper?

CHRYSANTHOU: We have no trouble with dust at the chopper. We have dust at the top at the bin after the material has been broken down in the pipe during conveyance.

QUESTION: Is the tunnel pressurized?

CHRYSANTHOU: No, we never pressurize.

QUESTION: Where is the pressure applied?

CHRYSANTHOU: Well it's fed at the bottom to the stower and the stower is, if you can picture it, a paddle wheel rotating at 40 rpm. The material will drop into it and all the paddle wheel will do is feed it into the line where the air is coming in from the blower.

QUESTION: What pressure do you use in there?

CHRYSANTHOU: Our maximum is 18 psi, but we are working with about 8 or 9 psi right now.

QUESTION: Did you investigate other kinds of pipe?

CHRYSANTHOU: We entered into a covenant to purchase that material from Ragmar Engineering under rental-to-purchase basis. This meant that they had the right to set the pipe and the equipment on the job. The pipe is made in West Germany. It's made by Essen and to date has been an excellent purchase, an excellent choice.

QUESTION: You mention your best day using this method was 55 ft. How does that compare to your normal progress?

CHRYSANTHOU: It's about 15 ft. less.

QUESTION: Have you experimented with the smaller size pipe than the 10 in.?

CHRYSANTHOU: No. This has been our first application. Of course, as you know, that the diameter of the pipe is a function of the size of the material that you're going to carry through it, the velocity its going to travel through it, and the air it should take. Ten in. happened to be in our calculations, the minimum size we could use in this type of application.

QUESTION: Is the air completely contained from the time it goes in till the time it comes back out?

CHRYSANTHOU: Yes sir.

QUESTION: What are the abrasive characteristics of Edmonton formation compared, for exampled, to the limestone that we have in the Chicago area?

CHRYSANTHOU: I don't think it compares. I think that our abrasive factor is very very low down there with our shales. We have never tested it, but we know that the life of our tungsten carbite bits is quite high; so really we don't have an abrasion problem there at all.

QUESTION: I'm interested in the comment that you're doing this work with about six of your own crews headed by a force account.

CHRYSANTHOU: That's a loaded question. What do you mean by force account?

QUESTION: In other words, your not bidding the job.

CHRYSANTHOU: Well, let's put it this way. We put an estimate in and everybody is willing to take a crack at it if he wants to.

I think we have a lot of contractors here. To do them justice I should explain to them that in 1955, when we started initiating this program of sewer construction, we set the first tunnel to tender. Of course, not having any contractors that were experienced in tunneling in Edmonton at the time, they went into joint ventures with some of the American contractors. The price was so exorbitant for an 8 ft. diameter tunnel that we felt that we could do the job ourselves and write off the capital investment against one project, which we did. Then we kept on growing and growing till it's too late to get out. But, basically as a member of a form of government I'll tell you that my primary interest is to get the cheapest price I can get for the City of Edmonton. Now whether I do the job, or whether a contractor does it is immaterial and irrelevant. As long as he can prove to me that he can do it cheaper, more power to him.

QUESTION: There's another part of this question. What kind of help are you using to run the machines? In other words, are you using operating engineers, hoisting engineers or laborers?

CHRYSANTHOU: No. Our system in Canada is a little different than yours is. An employee who works for a municipal government in Edmonton joins a union that's called the Canadian Union of Public Employees. You have your welders, mechanics and tunnel diggers, maintenance workers, sewer maintenance workers and the fellow that cuts the grass in the park. The are all lumped in one union. It makes it easier to deal with, a lot easier.

QUESTION: Is there a patent on any of the features of the pneumatic system?

CHRYSANTHOU: I think that you would have to get in touch with Ragmar Engineering in Portland. I couldn't answer that.

QUESTION: Does the air come from the compressor or a blower?

CHRYSANTHOU: The blower.

QUESTION: What was the removal rate of the material?

CHRYSANTHOU: About 200 tons an hour.

QUESTION: How many miles of tunnel have you dug since 1955?

CHRYSANTHOU: I must have about 160 mi. under my belt now.

Section 9

GEOLOGIC EXPLORATION FOR CHICAGOLAND AND OTHER DEEP ROCK TUNNELS TO BE CONSTRUCTED BY MECHANICAL MOLES

by

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GEOLOGICAL EXPLORATION FOR THE CHICAGOLAND DEEP TUNNEL PROJECT AND OTHER ROCK TUNNELS TO BE CONSTRUCTED BY MECHANICAL MOLE

PURPOSE AND SCOPE

The Chicagoland Deep Tunnel Project of the Metropolitan Sanitary District of Greater Chicago was proposed as a method to reduce flooding and pollution caused by the overflow of the combined sanitary and storm water sewer system during periods of heavy rainfall. The details of the Deep Tunnel Project are discussed in the papers included in these proceedings by Keifer, Koelzer, and Neil.

In general, as shown in Fig. 1, the Deep Tunnel concept consists of interception of the combined sanitary and storm water overflow at the overflow points, conveyance of the overflow water in tunnels to a mined room and pillar type storage area from which the overflow water can be pumped at a reduced rate to permit treatment of all waste water. As an added feature, water can be stored in a surface reservoir and released to the lower reservoir to provide the capability of peak power generation.

The purpose of this paper is to describe the subsurface geological exploration program performed in 1967 and 1968 for the Chicagoland Deep Tunnel Project. This program illustrates the type and magnitude of exploration performed to demonstrate the technical feasibility of the project.

The Deep Tunnel Project was divided into the Master Plan area and the First Construction Zone as shown in Fig. 2. The Master Plan Area included the total project area and the First Construction Zone was the first area being considered for actual construction. The papers by Keifer and Neil present later modification to the original construction plan.

GEOLOGIC CRITERIA

The geologic criteria required for the Deep Tunnel Project as set forth in earlier planning studies included the following:

1. Rock strata of adequate thickness for the tunnels or the mined storage area.
2. Rock capable of providing long term stability with a minimum of supports.

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3. Uniform rock characteristics which are desirable for excavation by mechanical moles.
4. A minimum of water problems from groundwater inflow.
5. Protection of the groundwater resources from contamination which could be caused by outward seepage of the storm water from the tunnels or the mined storage reservoir.

GEOLOGIC SETTING

The First Construction Zone is located on the east flank of a broad structural arch known as the Kankakee Arch. This arch separates the Illinois and Michigan structural basins. Because of erosion along the crest of the arch, some of the strata present in eastern Cook County are absent farther west. The eastward dip of the strata on the eastern side of the arch in the Chicagoland area is generally 15 to 25 ft. per mile. Superimposed on the regional dip are a series of secondary folds whose axes trend approximately east-west.

The substantiated faulting in the Chicagoland area is believed to have occurred after the deposition of the Silurian strata. Previously reported faults include: the Sandwich fault zone, near Joliet which trends southeast and has a displacement over 1,000 ft.; a complexly faulted area at Des Plaines, Illinois; and several small northeast trending faults reported in Chicago Avenue water supply tunnel constructed in the late 1920's and early 1930's. In the First Construction Zone several faults have been interpreted on the maps prepared by Seismograph Service Corporation.

The strata in the First Construction Zone consists of both soil and rock deposits. The soil deposits are made up of artificial fill and Pleistocene age deposits of lacustrine clays and sands, glacial till, and outwash sands and gravels. The rock strata are made up of a thick sequence of Silurian, Ordovician, and Cambrian Age sediments consisting of dolomites, shales, and sandstones. The deposition of the sedimentary strata was interrupted at various times and several erosional unconformities are present.

SUBSURFACE EXPLORATION PROGRAM

PURPOSE

A four phase subsurface exploration program was performed to demonstrate the technical feasibility of the Chicagoland Deep

Tunnel Project. The exploration program included seismic surveying, geophysical borehole logging, test drilling and laboratory testing, and groundwater drilling and testing. Seismic surveying and geophysical borehole logging were performed in the Master Plan Area and all four phases of exploration were performed in the First Construction Zone.

SEISMIC SURVEY

PURPOSE

The seismic survey was performed (1) to establish the bedrock topography, (2) to establish the configuration of the top of the Galena Group, and (3) to locate potential faults.

SCOPE

The seismic survey was laid out on a basic 4 mile grid spacing in the Master Plan Area and on a basic 2 mile east-west spacing and 4 mile north-south spacing in the First Construction Zone. Minor modification in the basic layout was made to accomodate variations in the city street system and to obtain more detail in subsurface information in certain areas. The location of seismic lines is shown on Fig. 3.

Approximately 420 miles of seismic traverses were made during the periods of November, 1967 to March, 1968 and May, 1968 to June, 1968.

FIELD METHODS

Seismic exploration methods have been used for many years to determine subsurface rock conditions and attitudes. These techniques have largely been confined to the search for oil or to measurement of the earth's crustal structure on a continental basis. The method rarely has been applied to construction problems on a broad scale.

There are two variations of seismic surveying techniques based on physical phenomena common to the physics of wave motion: the refraction and reflection methods. Fig. 4 and 5 provide diagrammatic illustration of the two diverse techniques. Both utilize a controlled source of energy and measure the travel time through the rock layers to suitable receptors on the surface. The travel times can be converted to depth measurements, and the configuration of the surface of a stratum can be interpreted by contouring.

In the refraction method, travel times are measured for energy transmitted through the rock layers for an extended horizontal distance as compared to the vertical components of its path. In the reflection method, near vertical travel of the energy is measured as it is reflected from various subsurface rock layers back to the surface. Specific reflective interfaces occur where adjacent rock layers exhibit dissimilar acoustic properties.

In the Chicagoland area, it was desired to map the bedrock surface and the attitude and continuity of the deeper Galena-Platteville strata. The shallow depths to bedrock, generally less than 150 ft., suggested the refraction technique as the most efficient means of obtaining the near-surface information. Since the relative acoustic velocities made it virtually impossible to secure information from the deeper rock layers by refraction means, the reflection method was employed for these measurements.

Because of the intense cultural development in the Chicagoland area, the use of explosives for an energy source was not acceptable. Therefore, the Vibroseis* method of seismic exploration, which has features that particularly adapt it to use in an urban environment, was selected. The Vibroseis system consists of recording energy from a vibratory source and subsequently applying correlation methods to reduce the data to geologically interpretable results. The truck-mounted vibrators (Fig. 6) introduced a signal of several seconds duration into the earth. The seismic signal has frequency characteristics which vary progressively over a predetermined range. Knowledge of the form of this input signal enables extraction of the signal from a background of very high ambient noise when processing the recorded data. In the Chicagoland area, random noise of considerable magnitude was generated by vehicular traffic and industrial operation. Nevertheless, the Vibroseis system permitted operations to be conducted in daylight hours, with consequent improvement in efficiency. During peak traffic conditions, where the presence of the equipment and personnel on the principal thoroughfares would have contributed to traffic congestion, work was suspended. The Vibroseis signals had an additional attribute in that they in no way disturbed persons or structures in the vicinity.

Two Vibroseis field crews were utilized simultaneously to conduct most of the investigation. Each field crew was comprised of: three truck-mounted vibrator units which operated in synchrony; truck-mounted electronic instrumentation for receiving and recording the seismic signals; and service vehicles to handle the positioning of individual detectors and their connecting cables, for the maintenance of equipment, and for the transportation of personnel. Radio communication was maintained between the various units involved in the operation.

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*Trademark and service mark of the Continental Oil Company

DATA PROCESSING

A flow diagram illustrating the acquisition and processing of the seismic data is shown in Fig. 7. The management of the operation, the initial processing of the data, and the preliminary interpretation of the results were handled from a centrally located project headquarters in Elmhurst, Illinois. The individual crew offices were maintained in the immediate area of operations and were moved from time to time to minimize non-productive travel time. Final processing and interpretation of the information was done utilizing digital computing equipment and facilities at the contractor's plant in Tulsa, Oklahoma.

Seismic cross sections were prepared in terms of travel time and were then subjected to interpretational evaluation to establish identification of the geologic horizons represented by the reflections, and to determine lithologic continuity and location of zones of possible faulting associated therewith.

The final stage of the interpretive process consisted of conversion of the various time measurements to depth values and the subsequent display of this information on digitally plotted depth cross sections and on contoured maps. To perform this conversion, knowledge of the speed of travel of the energy through the various rock layers was necessary. Such velocity information was obtained by comparison of the seismic data with geophysical borehole information obtained from water wells adjacent to the lines of seismic traverse and from deep holes drilled specifically for the Deep Tunnel Project. Direct borehole velocity measurements were made in one test hole to provide positive confirmation of the identification of the reflecting horizons. The velocities established for the various rock units are shown on Fig. 8.

RESULTS

Good correlation was obtained between the seismic data and the horizon tops reported in most of the wells. However, for each horizon mapped, the velocity through the overlying material was found to vary across the area on a geographic basis. Consequently, iso-velocity maps were prepared for each seismic event mapped, and each seismic time datum value was converted to a depth relative to sea level according to the appropriate velocity for its geographic location, before being placed on the graphic displays.

The results of the seismic survey were presented in the form of contour maps and cross sections. The maps are based on all available borehole data and on the calculated seismic depths. They include (1) the top of rock, (2) the top of the Galena Group, (3) a partial interpretation of the top of the Ancell Group, and (4) an isopach map of the interval between the top of rock and the top of the Galena Group. The top of rock and the top of the Galena were of primary importance in this study and the discussion is limited to these surfaces.

Cross section based on continuous seismic depth calculations were made for each seismic traverse. The cross sections show (1) the ground surface, (2) the top of rock, (3) the top of the Galena Group, (4) the top of the Ancell Group where obtained, and (5) at a few locations, the top of an unidentified Cambrian Age formation.

Top of Rock. The bedrock in the Chicago area is overlain by a variety of materials including: man-made fills; glacial deposits of till and outwash sands and gravels; and lacustrine deposits of sands, silts and clays.

The bedrock surface is the result of a complex geologic history which included folding, faulting and erosion. The erosional history had the dominant affect on the bedrock surface and, therefore, it is not possible to clearly define folds or faults.

The complexity of the soil deposits which overlie the bedrock, the complex nature of the bedrock surface, and the shallow depth of the bedrock were major problems which had to be considered in the seismic interpretation. The resulting contour maps depicting the bedrock topography are believed to present an interpreation with the probable error not greater than ±10 ft. where the seismic data are recorded.

Top of Galena. The configuration of the top of the Galena Group is the result of folding and faulting. The interpretation of this surface in the First Construction Zone is shown on Fig. 9. The accuracy of this interpretation is believed to be ±25 ft. where the seismic data are recorded. The regional dip is to the east, but a series of minor folds are superimposed on the regional dip making the Galena surface somewhat more complex.

The seismic data indicates the possibility of nine faults which cut the top of the Galena in the First Construction Zone (Fig. 9). The calculated displacement along these faults ranges from less than 10 ft. to as much as 55 ft.

Seismic exploration is an indirect method of exploration and the faults shown on Fig. 9 have not been substantiated by direct exploration methods. It is believed, however, that the seismic surveys will detect all potential faults having vertical displacements of 20 ft. or more and in some instances will detect those with displacements as low as 10 ft.

Therefore, the method is valuable to locate potential faults, but their actual presence must be substantiated by direct subsurface exploration techniques.

GEOPHYSICAL BOREHOLE LOGGING

PURPOSE

The broad purposes of the geophysical borehole logging program were twofold: (1) to establish the geophysical characteristics of the stratigraphic units and (2) to determine the variations in the rock characteristics of each of the stratigraphic units. The logging tools selected for use on this project were chosen for specific purposes within this broad framework. Logs were required which would (a) supply correlations from hole to hole, (b) define certain rock properties such as: porosity, fluid content and movement, and engineering characteristics such as density and the elastic moduli and, (c) log data which would supplement and corroborate the seismic data.

SCOPE

A total of 41 boreholes were logged geophysically. Of these, 11 were existing wells in the area which were logged to provide a wider distribution of stratigraphic data, 19 were rock holes drilled and cored for the Deep Tunnel Project, 10 were ground water test holes drilled, but not cored for the Deep Tunnel Project, and 1 soil exploratory hole drilled in the upper reservoir area.

LOGGING TOOLS AND METHODS

The logging program included the following logs: Gamma Ray, Neutron, Formation Density, 3-D Velocity, Temperature and Caliper.

Gamma Ray Log. Gamma Ray logs measure the natural radioactivity present in rocks in fluid-filled and dry holes, whether open or cased. Shales normally contain more of the three most commonly occurring radioactive elements (uranium,potassium and thorium) than do other sedimentary rocks, so the Gamma Ray log may be considered as a shale identifying log. Use of the log permits evaluation of the shaliness of limestones, dolomites, and sandstones. The Gamma Ray log is sensitive to hole enlargement, the presence of casing and of borehole fluid. The effect of all of these conditions, however, can be corrected.

Neutron Log. The Neutron log measures the amount of hydrogen contained in rocks. The hydrogen content is directly related to porosity so the Neutron log becomes a porosity measuring device. The Neutron probe contains a source which continuously bombards the rock formation with a cloud of high energy neutrons. These neutrons collide with nuclei of matter in the borehole and surrounding formation and are ultimately captured. If they collide with heavy atoms of elements like silicon, aluminum, iron, calcium and oxygen, they will be reflected elastically without much loss of energy. If they collide with light hydrogen atoms, which have almost the same mass as neutrons, their energy will be greatly reduced. When the energy is reduced to a certain level by continued collisions, the neutron is captured and gamma rays of capture are emitted. These gamma rays of capture are measured by a detector spaced at a fixed distance from the source. Where hydrogen is confined to water present in pores, the Neutron log then measures porosity. Shale also contains an abundance of hydrogen as combined water. The Gamma Ray log may be used to correct the shale influence on the Neutron log. Hole enlargement, casing, or a change in fluid content in the borehole will affect the Neutron log, but again, these are all conditions for which correction can be made.

Formation Density Log. The Formation Density log measures the bulk density of rock. The probe in this tool contains a gamma ray source which continuously emits gamma rays into the borehole wall. The emitted rays bounce off electrons and the intensity of the reflected rays as seen by a detector in the probe is dependent upon the electron density. Electron density is directly proportional to the bulk density of all rocks of interest here. The density log is another porosity tool, and density is one of the factors used in combination with velocity data to compute the elastic moduli.

3-D Velocity Log. The 3-D Velocity Log measures the transmission time of sonic energy in both the compressional and transverse modes through the rock surrounding a borehole. The logging probe consists of magnetostrictive transmitter which is pulsed approximately 20 times per second as the probe is moved upward in the hole. The pulse energy moves outward through the borehole fluid and at the borehole face is refracted and travels in the rock media. It is detected as it moves past the receiver. The various modes of signal travel, which propagate at different velocities, are recorded as a variable intensity display. Fig. 10 shows a section of a 3-D Velocity log with the pressure, shear, and boundary wave arrivals indicated.

The 3-D Velocity log supplies the basic data for the computation of the engineering properties of rocks: bulk, shear, and Young's moduli, and Poisson's ratio. Since it measures compressional wave velocity, it is also useful in converting seismic survey time data to depths. The seismic system is dependent on velocity and/or density changes at which reflection of the input energy may occur. The velocity and density logs both show graphically the interfaces which should reflect energy.

The 3-D logger is sensitive to changes in borehole diameter and can only be used in fluid-filled holes.

Temperature Log. The temperature Log measures the temperature in fluid-filled and dry holes, and in open or cased holes. The downhole probe contains a sensitive, quick-reacting thermistor. Under normal increases with depth in accord with the geothermal gradient existing in the area. When the temperature curve deviates from the normal gradient, it indicates a zone in which fluid is entering or leaving the hole and the approximate volume of flow.

Caliper Log. The Caliper Log is used to supply a record of changes in borehole diameter primarily for use in interpreting other logs which are sensitive to such changes. There are several types of calipering devices, but the most commonly used has 3 pencil-like arms at 120° lateral separation. These measuring arms are able to detect small, thin changes such as washed out shale laminations.

All of the foregoing logs are run from a unit as shown in Fig. 11. Hoisting equipment and cable reels occupy approximately half the cab space, while the other half houses the signal processing equipment and recorders.

RESULTS

An example of the geophysical borehole logging results is shown in Fig. 12. The Joliet Formation and the Guttenberg Formation were found to be two geophysical marker beds in the Chicago area. The Racine Formation was found to be a variable dolomite, the Joliet Formation a fairly uniform dolomite, the Kankakee Formation has numerous shale partings, the Brainard Formation is a fairly uniform shale, the Fort Atkinson Formation is a fairly uniform dolomite, the Scales Formation is a fairly uniform shale, and the formations in the Galena and Platteville Groups are fairly uniform dolomites.

Borehole logging results from the cored holes were used (1) to correlate the stratigraphy from cored holes to the previously existing boreholes which were geophysically logged during this project, and (2) to establish the stratigraphy in the ground water test holes so packers could be set to isolate hydrologic units.

TEST DRILLING

PURPOSE

The purpose of the rock core drilling program was (1) to provide cores from which the rock characteristics of the various strata could be evaluated so the most favorable elvations could be selected, (2) to provide positive stratigraphic control in key areas, (3) to provide core for correlation with the geophysical borehole logs, and (4) to provide boreholes in which the water bearing properties of the rock could be evaluated and monitored.

SCOPE

Twenty-four core holes with a total footage of 34,360 ft. were drilled during the period from November, 1967 to March, 1968. The deepest hole, having a total depth of 1,696 ft., was drilled to the upper portion of the Eau Claire Formation.

FIELD METHODS

All rock core holes were drilled with wireline equipment and excellent results were obtained. Water pressure tests were run in all core holes to evaluate the water bearing characteristics of the rock strata. All the boreholes were instrumented with piezometers to obtain more accurate ground water data in the First Construction Zone.

RESULTS

Water pressure tests indicated, in general, that the dolomite strata under consideration had very low permeabilities and that large water inflows into tunnels or into the storage chamber would apparently be limited to fractured zones that may be present.

Excellent correlation was obtained between stratigraphic logging and geophysical borehole logging as shown on Fig. 12.

Laboratory analyses were performed on selected rock samples to establish the general range of properties of the various strata. The following tests were performed: specific gravity, unconfined compressive strength, modulus of elasticity (static and dynamic), drillability, natural water content, absorption, abrasion, porosity, permeability, petrographic analyses, x-ray analyses, chemical analyses, wetting and drying, and solubility. A partial summary of field and laboratory data is presented in Fig. 13.

The evaluation of the various rock strata based on physical examination, geophysical borehole logs, and laboratory analyses indicated the following formations best fulfilled the geologic criteria: Waukesha Joliet, Wise Lake, Dunleith, Guttenberg, Nachusa, Grand Detour, and Mifflin.

GROUND WATER DRILLING AND TESTING

PURPOSE

A detailed ground water testing program was performed (1) to determine the hydraulic characteristics of the hydrogeologic units shown on Fig. 14, (2) to determine the feasibility of aquifer protection by artificial recharge methods, and (3) to provide ground water observation wells.

Ten water wells with a total footage of 11,800 ft. were drilled during the period from November, 1967 to March, 1968. The wells included one deep aquifer test well, three observation wells, and six specific capacity wells.

FIELD METHODS

The water wells were drilled by the cable tool method and the air rotary method. After completion of the drilling, each well was geophysically logged to establish the stratigraphy so packers could accurately be set and the selected hydrogeologic units isolated and tested. A total of 20 pumping tests and two recharge tests were conducted in the specific capacity wells and in the deep aquifer test well.

Specific Capacity Wells. The Silurian aquifer system and the Galena-Platteville hydrogeologic unit were tested in each of the six specific capacity wells. Each well was drilled and cased through the overburden and then drilled through the Silurian to the top of the Maquoketa. The Silurian aquifer system was then pump-tested. After testing, the well was drilled to the top of the Galena and cased through the Maquoketa. Drilling was then continued to the top of the Glenwood. A packer or a concrete plug was set at the Platteville-Glenwood contact and the Galena-Platteville unit was pump tested.

Deep Aquifer Pump Testing. Seven pumping tests were performed in the deep aquifer test well. These tests were designed to evaluate the Silurian aquifer system, the Cambrian-Ordovician aquifer system, the Galena-Platteville hydrogeologic unit, the Glenwood-St. Peter hydrogeologic unit, the Prairie du Chien-Eminence-Potosi hydrogeologic unit, the Franconia hydrogeologic unit, and the Ironton-Galesville hydrogeologic unit.

The deep aquifer test well and the observation wells were drilled and cased through the overburden and then drilled through the Silurian to the top of the Maquoketa. The Silurian aquifer system was tested and the wells were then drilled to the top of the Galena and cased through the Maquoketa. The wells were then drilled to the top of the Eau Claire Formation at a depth of approximately 1685 ft. Each hydrogeologic unit was isolated by inflatable packers and tested individually.

Recharge Tests. After completion of the deep aquifer pumping test, two recharge tests were conducted in the deep aquifer well. Drinking quality water was injected into the various hydrogeologic units and the build-up of water levels was observed in the observation wells.

The Galena-Platteville hydrogeologic unit was isolated from the underlying hydrogeologic units in the two observation wells by means of a cement plug. An observation pipe was placed through the cement plug in each observation well to permit measurement of the water level in the underlying hydrogeologic units.

The hydrogeologic units to be recharged were isolated in the deep aquifer test well by means of inflatable packers.

In recharge test 1, water was injected through the Galena-Platteville, Glenwood-St. Peter, Prairie du Chien, Eminence, and upper 80 ft. of the Potosi as shown in Figure 15 A. Recharge test 1 was continued for a period of about 10 days as shown in Figure 15 B. The rate of recharge averaged 243 gpm and the build-up in the recharge well was 125 ft. at the end of the test.

In recharge test 2, water was injected through the Galena-Platteville and Glenwood-St. Peter hydrogeologic units as shown in Fig. 16 A. Recharge test 2 was continued for a period of about four days as shown in Fig. 16 B. The rate of recharge was 60 gpm initially, but was decreased to 15 gpm because of overflowing of the recharge well. Even at the lower recharge rate the well overflowed in four days and the test was discontinued. The recharge rate for the four day period averaged 16 gpm.

The water levels in the two observation wells had not completely recovered from test 1 when test 2 was started as shown in Fig. 16 B. Complete recovery of the water levels from test 1 was not achieved prior to the start of test 2 because of contractual time restrictions.

RESULTS

The results of the specific capacity tests are presented in Table 1 and the results of the deep aquifer pumping tests are presented in Table 2.

Analysis of the recharge test results indicated the following:

1. The hydraulic characteristics of the St. Peter sandstone are too low to permit economical injection rates into this unit to develop the necessary ground water mound.

2. Recharge wells will have to be drilled into the Potosi Formation to obtain the required hydraulic characteristics to

develop the ground water mound necessary to protect the aquifer.

3. Recharge rates during the tests were lower than predicted on the basis of pumping test results. This is believed to have been caused by (a) chemical composition differences between the recharge water and the natural ground water, (b) temperature differences between the recharge water and the natural ground water, and (c) aeration of the recharge water.

The aquifer protection system, shown in Fig. 17, consists of maintaining a positive ground water head on all unlined structures to insure inward seepage rather than leakage of contaminated waters. The water table in the Silurian aquifer system is above all tunnels. Fig. 17 shows the current (1968) position of the Cambrian-Ordovician water level, the future water level without recharge, and the water level with recharge.

The ground water testing program demonstrated the feasibility of an artificial recharge system to protect the ground water resourses in the vicinity of the proposed project.

SUMMARY

To obtain the most favorable geological condition for deep tunnels to be constructed by mechanical moles, the tunnels should be located in structurally sound and uniform rock strata with a minimum of potential ground water problems. If the tunnels are designed to be unlined and to carry sanitary water, it is necessary to carefully evaluate the ground water conditions to assure this valuable resource from becoming contaminated by outward seepage from the tunnels.

SELECTED REFERENCES

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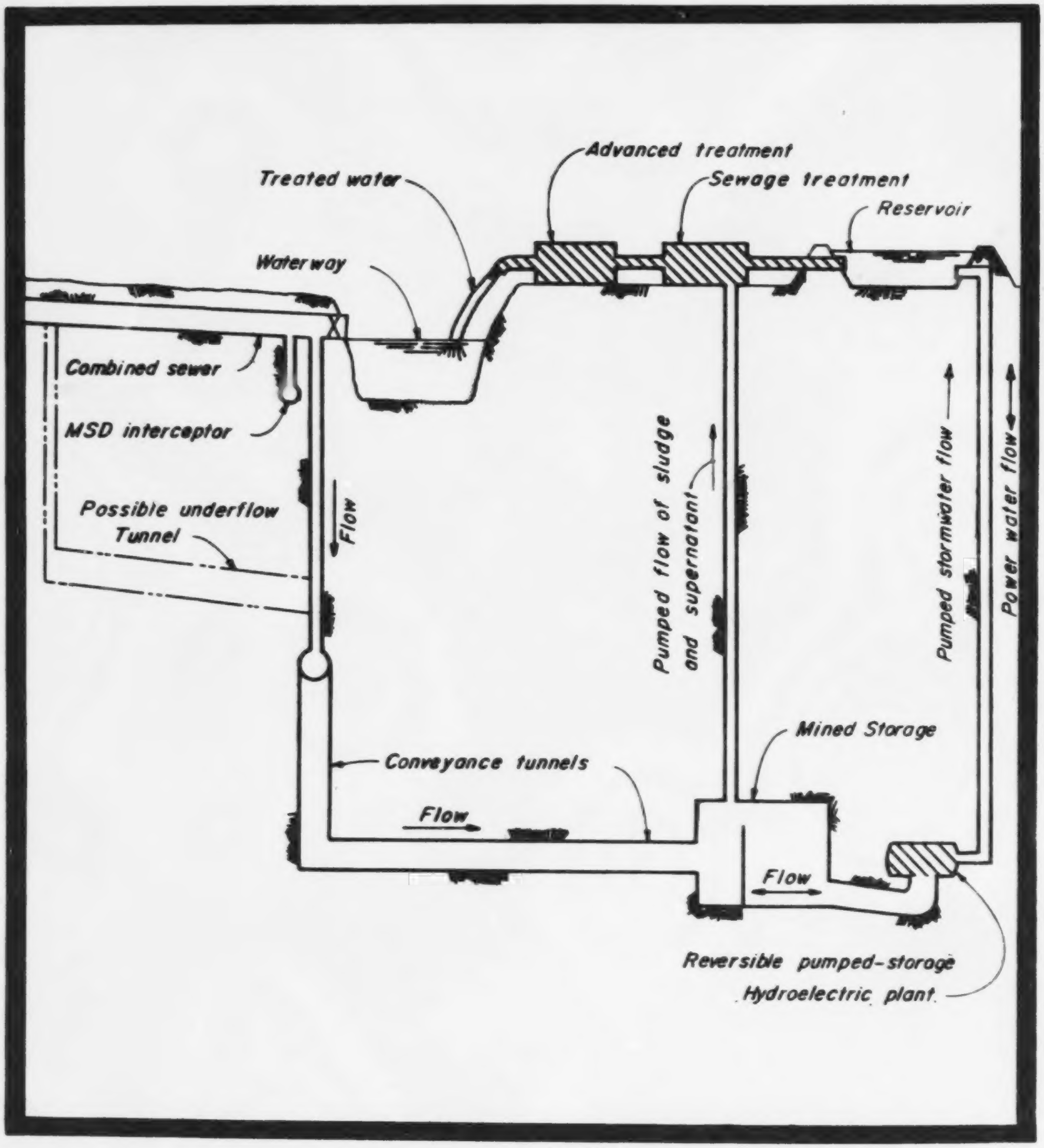
Harza Engineering Company and Bauer Engineering, Inc., 1968, Chicagoland Deep Tunnel System for pollution and flood control, first construction zone, definite project report, appendix G and H.

Harza Engineering Company and Bauer Engineering, Inc., 1969, The impact of the Deep Tunnel Plan on the water resources of northeast Illinois.

Seismograph Service Corporation, 1968, Report on a Vibroseis Survey for the Metropolitan Sanitary District of Greater Chicago, Chicagoland Deep Tunnel Plan for pollution and flood control;
    Phase I--mobilization and reconnaissance
    Phase II--first construction zone
    Phase III-total combined sewered areas

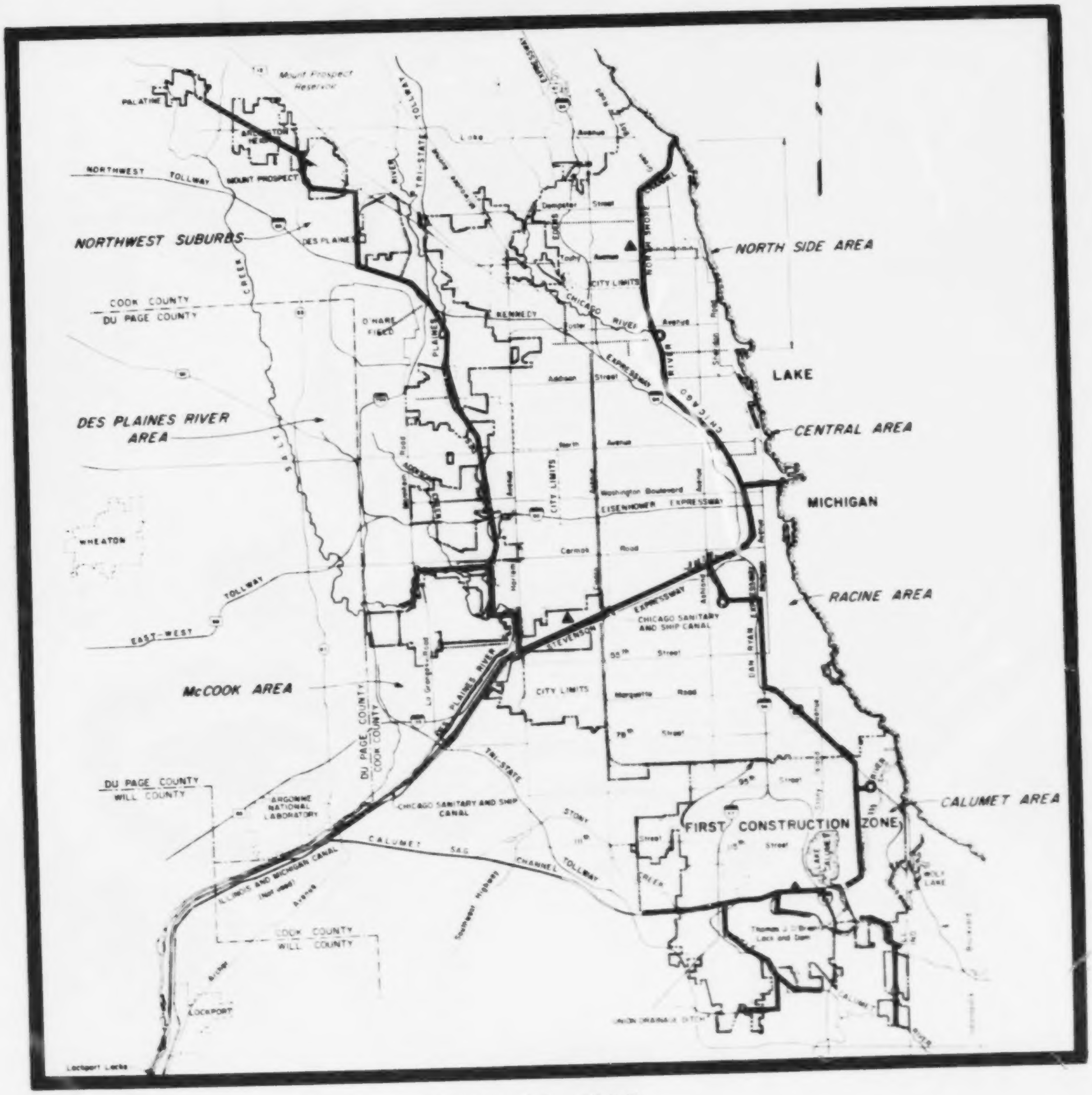
CREDIT FOR THE CHICAGOLAND DEEP TUNNEL PROJECT

Client	The Metropolitan Sanitary District of Greater Chicago Chicago, Illinois
Consulting Engineers	Harza Engineering Company Chicago, Illinois Bauer Engineering Inc. Chicago, Illinois
Geophysical Exploration	Seismograph Service Corporation Tulsa, Oklahoma
Borehole	Birdwell Division of Seismograph Service Corporation Tulsa, Oklahoma
Core Drilling	DiKor - Groves Carmi, Illinois
Ground Water Drilling and Testing	Layne-Western Company Aurora, Illinois


Advanced treatment
Treated water
Sewage treatment
Reservoir
Waterway
Combined sewer
MSD interceptor
Possible underflow Tunnel
Flow
Pumped flow of sludge and supernatant
Pumped stormwater flow
Power water flow
Mined Storage
Conveyance tunnels
Flow
Flow
Reversible pumped-storage Hydroelectric plant.

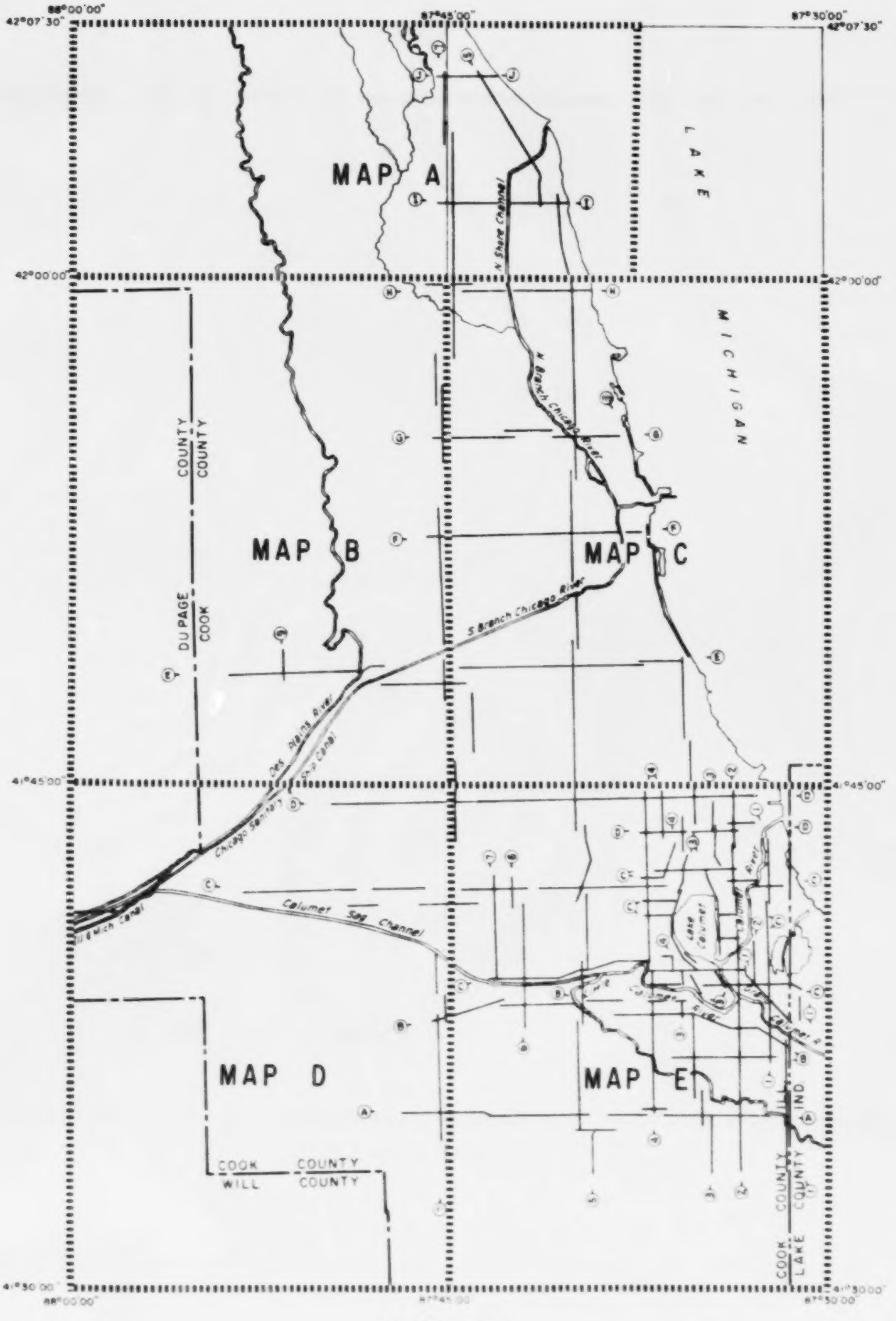
THE DEEP TUNNEL CONCEPT

Fig. 1


Mount Prospect Reservoir
PALATINE
ARLINGTON HEIGHTS
MOUNT PROSPECT
NORTHWEST TOLLWAY
NORTHWEST SUBURBS
DES PLAINES
TRI-STATE TOLLWAY
RIVER
Lake
Avenue
Dempster Street
EDENS
Touhy Avenue
CITY LIMITS
NORTH SHORE CHANNEL
NORTH SIDE AREA
CREEK
COOK COUNTY
DU PAGE COUNTY
O'HARE FIELD
PLAINES
KENNEDY
CHICAGO RIVER
Foster Avenue
EXPRESSWAY
RIVER
Sheridan Road
Addison Street
LAKE
CHICAGO
DES PLAINES RIVER AREA
CENTRAL AREA
SALT
North Avenue
CITY LIMITS
Washington Boulevard
EISENHOWER EXPRESSWAY
MICHIGAN
WHEATON
Cermak Road
Harlem
TOLLWAY
EAST-WEST
EXPRESSWAY
Ashland
CHICAGO SANITARY AND SHIP CANAL
RACINE AREA
STEVENSON
55th Street
CITY LIMITS
Marquette Road
DAN RYAN
McCOOK AREA
DU PAGE COUNTY
COOK COUNTY
DES PLAINES RIVER
La Grange Road
79th Street
TRI-STATE
95th Street
DU PAGE COUNTY
WILL COUNTY
ARGONNE NATIONAL LABORATORY
CHICAGO SANITARY AND SHIP CANAL
STONY
111th
CALUMET SAG CHANNEL
TOLLWAY
FIRST CONSTRUCTION ZONE
CALUMET AREA
115th Street
LAKE CALUMET
CREEK
WOLF LAKE
ILLINOIS AND MICHIGAN CANAL (Not Used)
Avenue
Southwest Highway
COOK COUNTY
WILL COUNTY
Thomas J. O'Brien Lock and Dam
LOCKPORT
UNION DRAINAGE DITCH
CALUMET RIVER
Lockport Locks

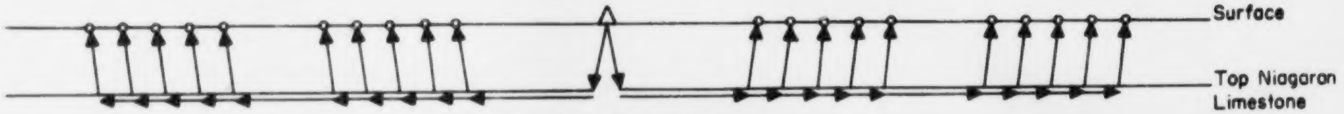
GENERAL MAP

Fig. 2


88°00'00"
87°45'00"
87°30'00"
42°07'30"
42°00'00"
41°45'00"
41°30'00"
MAP A
MAP B
MAP C
MAP D
MAP E
LAKE
MICHIGAN
N Shore Channel
N Branch Chicago River
S Branch Chicago River
Des Plaines River
Chicago Sanitary Ship Canal
Ill & Mich Canal
Calumet Sag Channel
Lake Calumet
Calumet River
DU PAGE COUNTY
COOK COUNTY
COOK COUNTY
WILL COUNTY
COOK COUNTY
LAKE COUNTY
IND

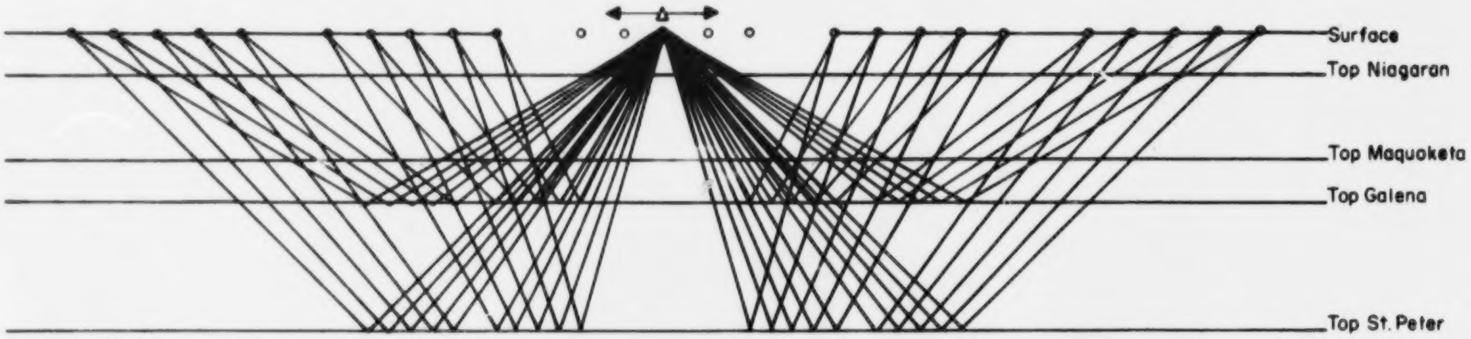
INDEX MAP

Fig. 3


Surface
Top Niagaran
Limestone

REFRACTION PROFILE

Fig. 4


Surface
Top Niagaran
Top Maquoketa
Top Galena
Top St. Peter

REFLECTION PROFILE

Fig. 5

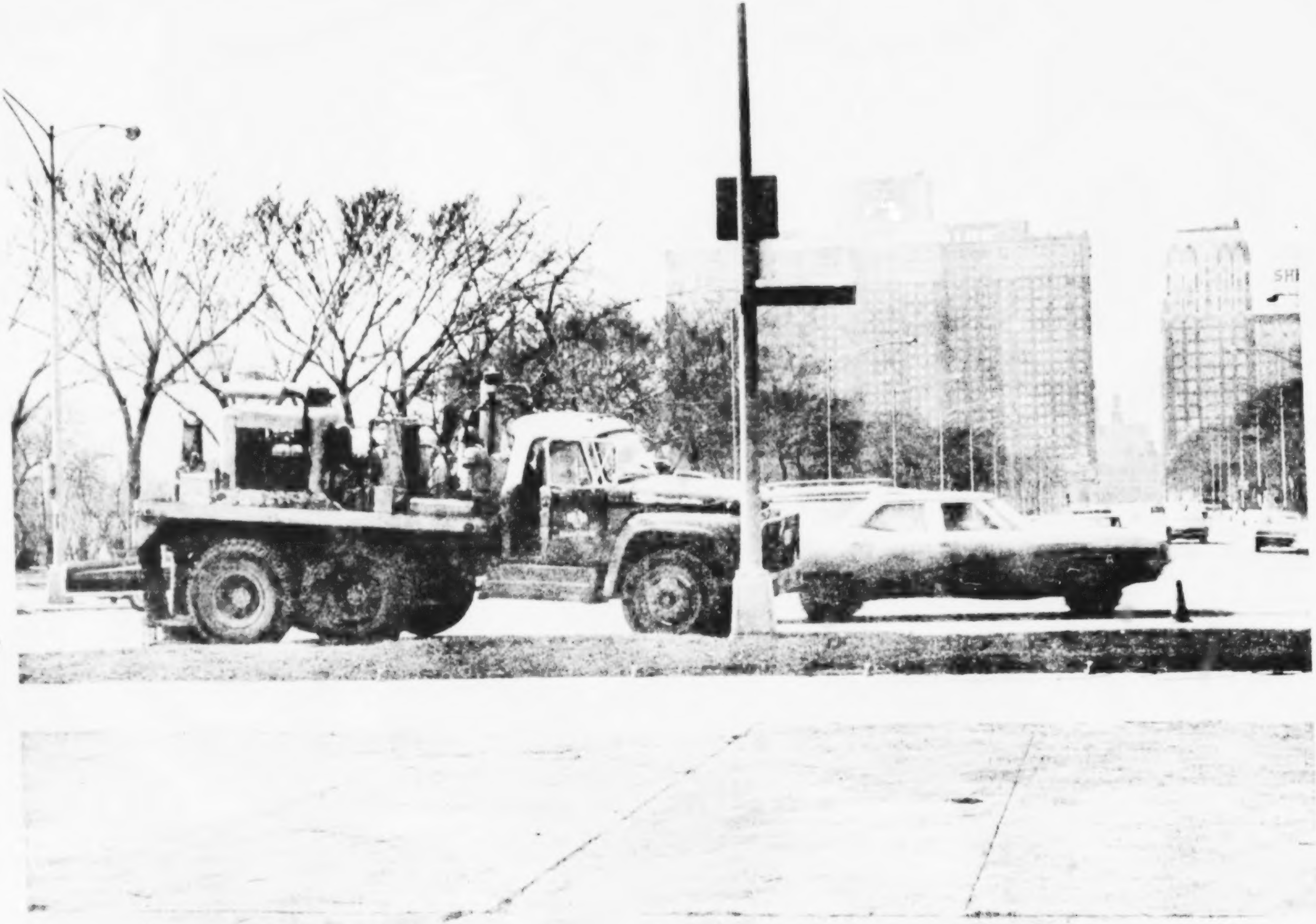


Fig. 6

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SEISMIC DATA PROCESSING FLOW DIAGRAM

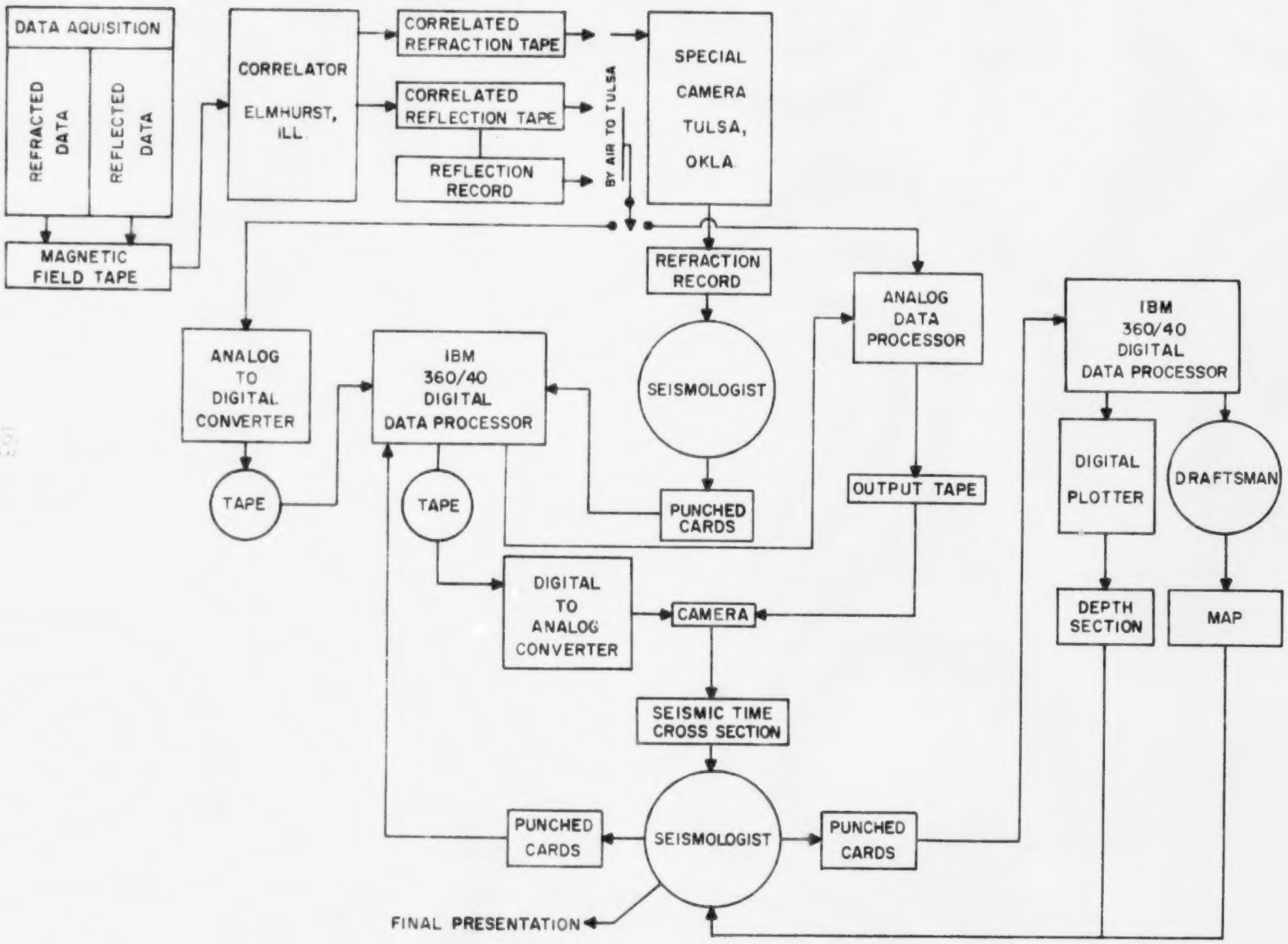
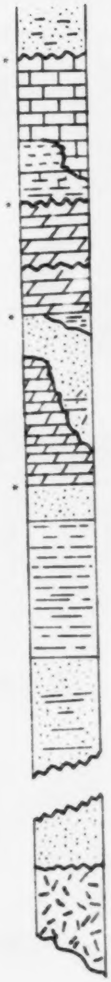

DATA AQUISITION
REFRACTED DATA
REFLECTED DATA
MAGNETIC FIELD TAPE
CORRELATOR
ELMHURST, ILL.
CORRELATED REFRACTION TAPE
CORRELATED REFLECTION TAPE
REFLECTION RECORD
BY AIR TO TULSA
SPECIAL CAMERA
TULSA, OKLA.
REFRACTION RECORD
ANALOG TO DIGITAL CONVERTER
TAPE
IBM 360/40 DIGITAL DATA PROCESSOR
SEISMOLOGIST
ANALOG DATA PROCESSOR
TAPE
PUNCHED CARDS
OUTPUT TAPE
DIGITAL TO ANALOG CONVERTER
CAMERA
SEISMIC TIME CROSS SECTION
IBM 360/40 DIGITAL DATA PROCESSOR
DIGITAL PLOTTER
DRAFTSMAN
DEPTH SECTION
MAP
PUNCHED CARDS
SEISMOLOGIST
PUNCHED CARDS
FINAL PRESENTATION

Fig. 7

GENERALIZED STRATIGRAPHIC COLUMN OF THE CHICAGOLAND AREA

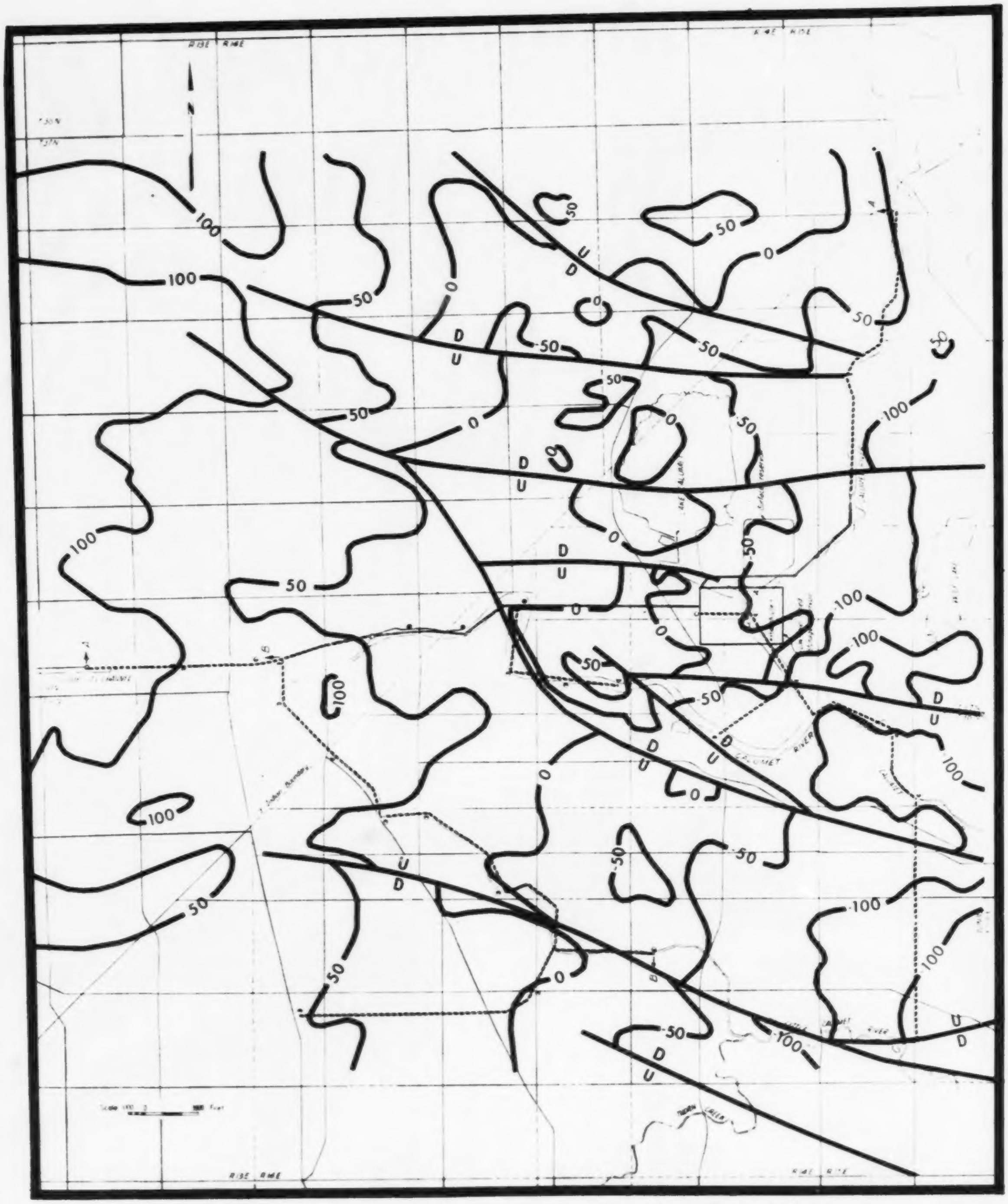


Thickness	System	Series, Group, or Formation	Lithology	Seismic Velocity
50'-150'	Quaternary	Glacial and lake deposits	Sand and clay	2000--8000 ft/sec
150'-300'	Silurian	Niagaran Series	Dolomitic limestone	16,500--19,000 ft/sec
20'-140'		Alexandrian Series	Dolomitic limestone	16,500--19,000 ft/sec
100'-250'	Ordovician	Maquoketa Group	Shale and dolomite	13,000--15,000 ft/sec
170'-225'		Galena Group	Dolomite	18,000--19,500 ft/sec
100'-150'		Platteville Group	Dolomite	18,000--19,500 ft/sec
0-80'		Glenwood Formation	Sandstone, shale & dolo.	13,000--18,000 ft/sec
100'-600'		St. Peter Formation	Sandstone	15,000--17,000 ft/sec
0-340'		Prairie du Chien Group	Dolomite and Sandstone	17,000--18,000 ft/sec
50'-150'	Cambrian	Eminence Formation	Dolomite	17,000--18,000 ft/sec
90'-220'		Potosi Formation	Dolomite	17,000--18,000 ft/sec
50'-200'		Franconia Formation	Sandstone, dolo. and shale	17,000--18,000 ft/sec
80'-130'		Ironton Formation	Sandstone	17,000--18,000 ft/sec
10'-100'		Galesville Formation	Sandstone	13,000--16,000 ft/sec
370'-575'		Eau Claire Formation	Shale, dolo. & sandstone	
1200'-2900'		Mt. Simon Formation	Sandstone	
	Precambrian		Granite	

* Seismic mapping level

Stratigraphic Column Modified From
T. C. Buschbach
and
H. B. Willman, Illinois State Geological Survey

Fig. 8


N
100
50
0
-50
100
50
0
U
D
D
U
CALUMET RIVER
LITTLE CALUMET RIVER

GEOLOGIC STRUCTURE – TOP OF GALENA
49 HOLES + SEISMIC

Fig. 9

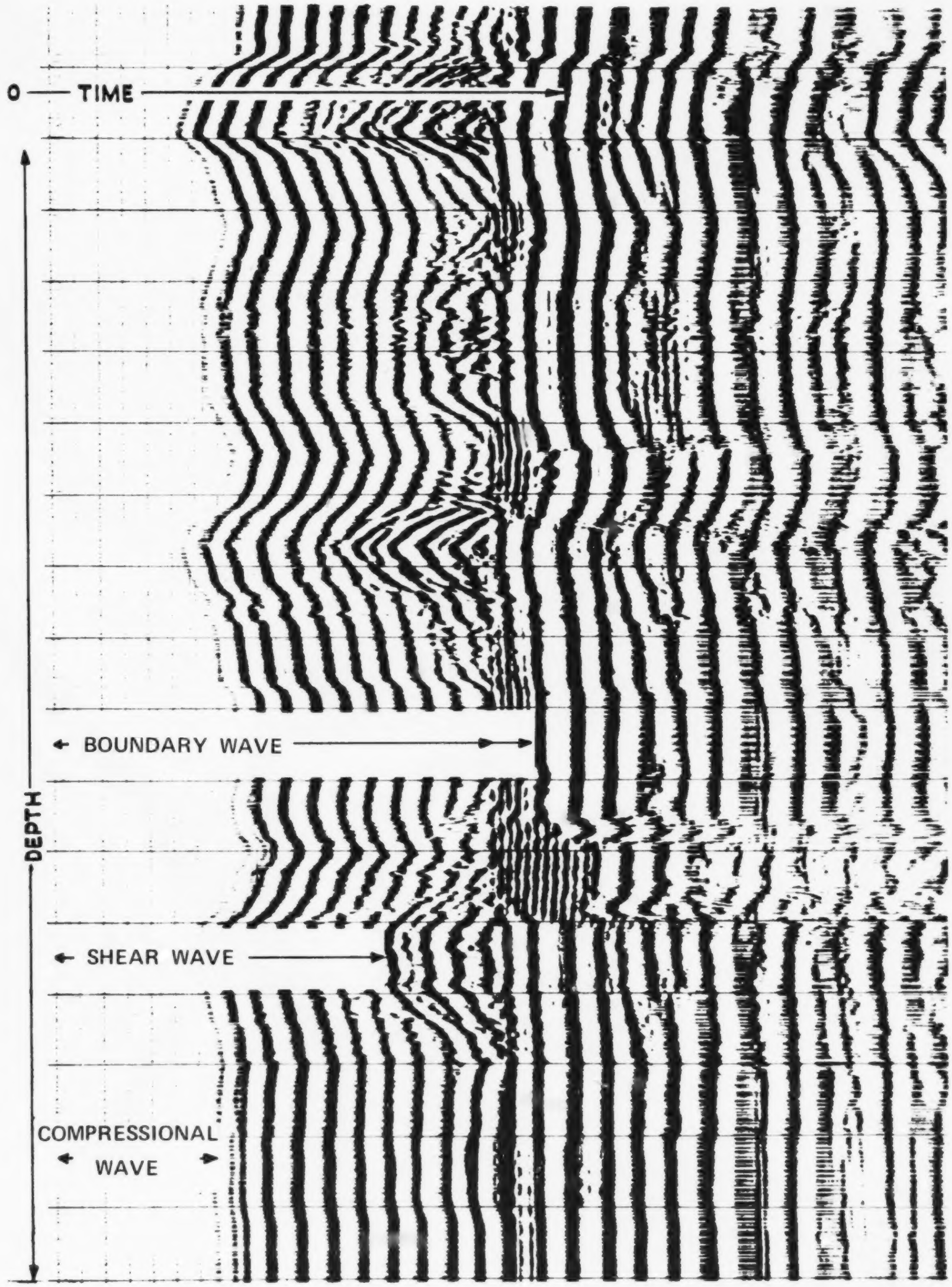


Fig. 10

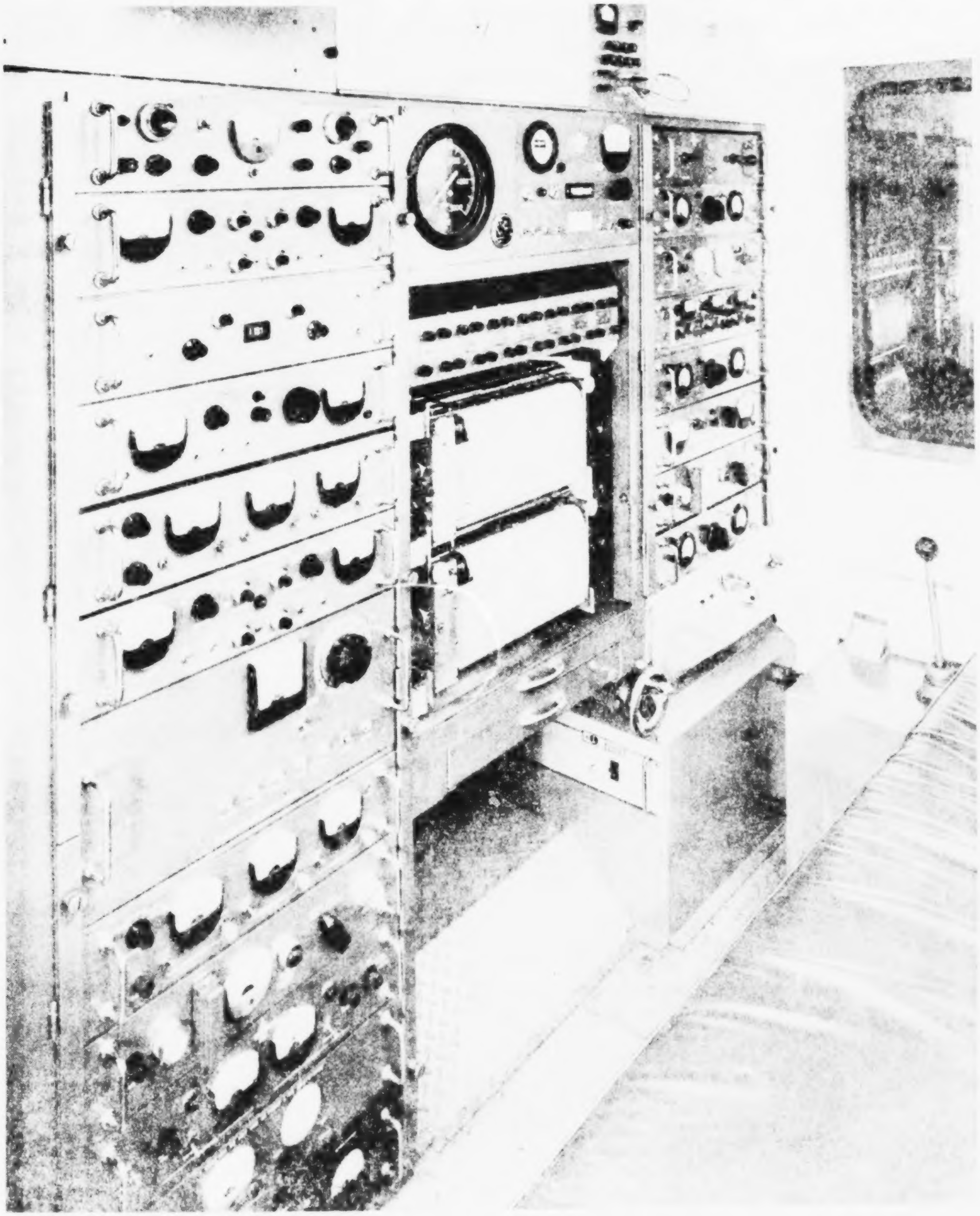


Fig. 11

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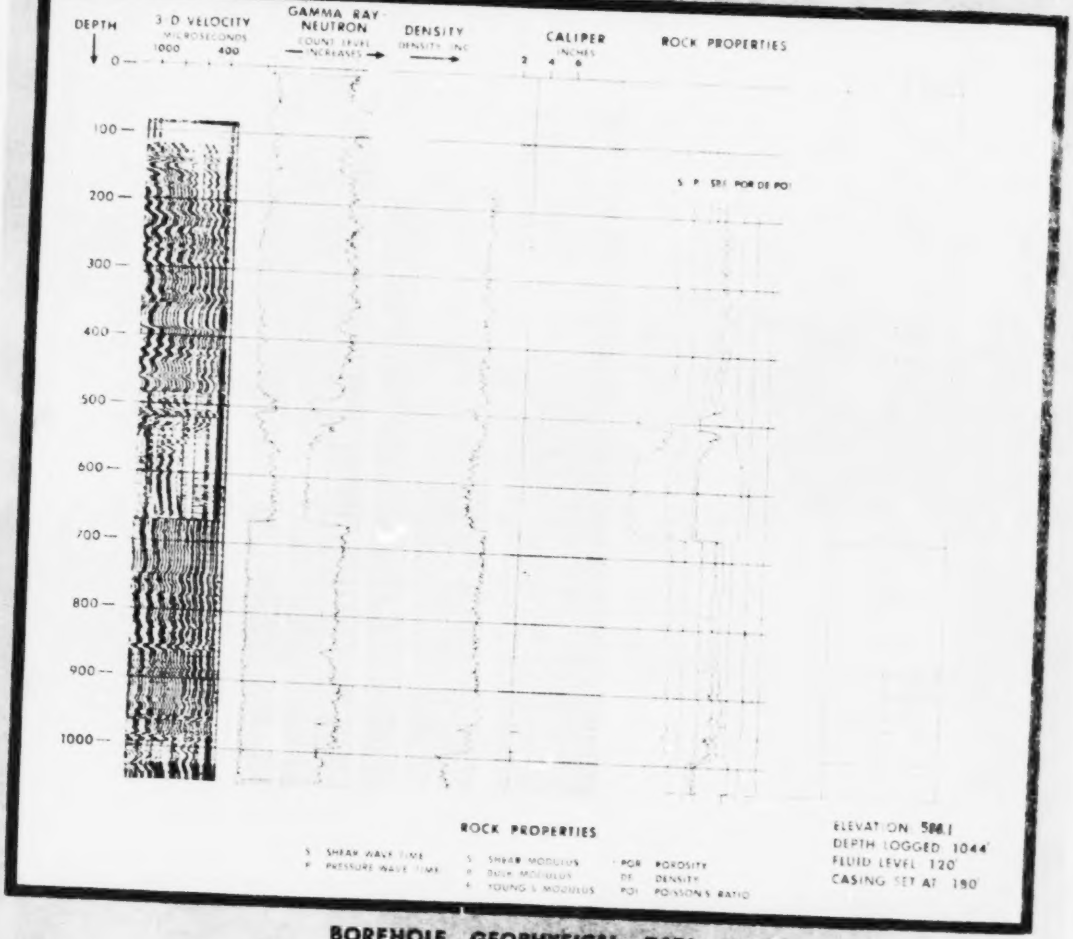
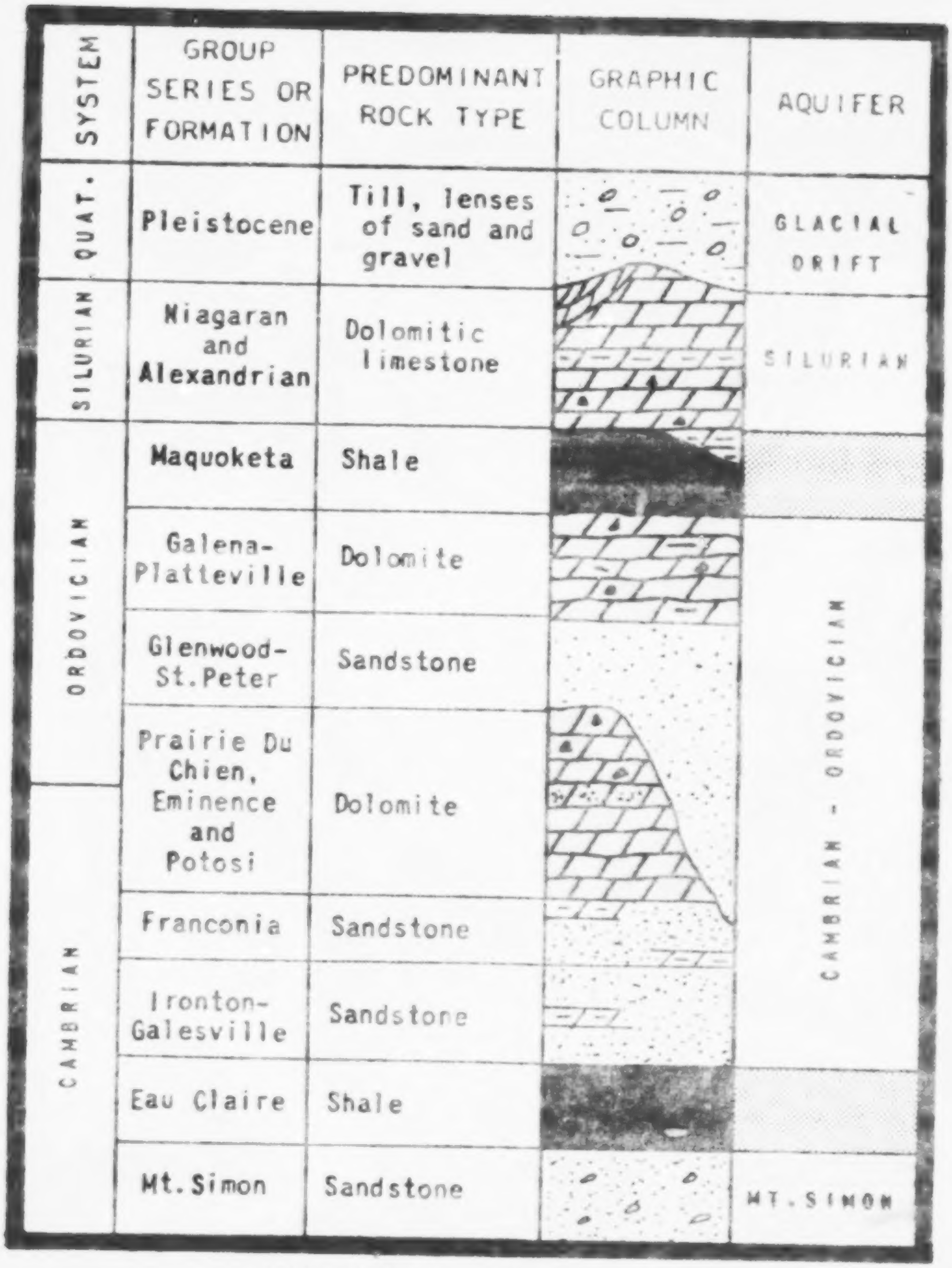


BOREHOLE GEOPHYSICAL DATA

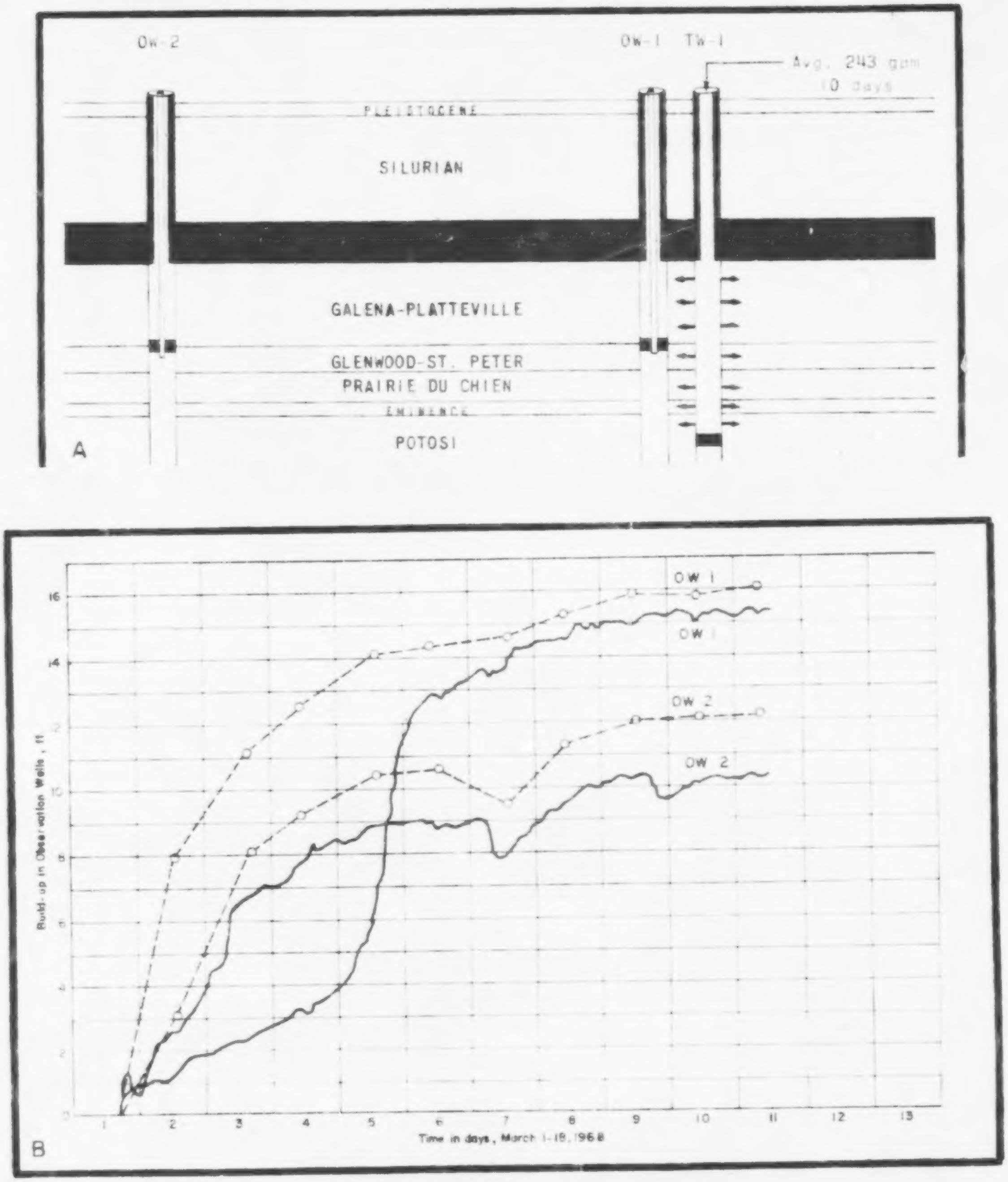
Fig. 12



SYSTEM	GROUP SERIES OR FORMATION	PREDOMINANT ROCK TYPE	GRAPHIC COLUMN	AQUIFER
QUAT.	Pleistocene	Till, lenses of sand and gravel		GLACIAL DRIFT
SILURIAN	Niagaran and Alexandrian	Dolomitic limestone		SILURIAN
ORDOVICIAN	Maquoketa	Shale		
	Galena-Platteville	Dolomite		CAMBRIAN - ORDOVICIAN
	Glenwood-St.Peter	Sandstone		
CAMBRIAN	Prairie Du Chien, Eminence and Potosi	Dolomite		
	Franconia	Sandstone		
	Ironton-Galesville	Sandstone		
	Eau Claire	Shale		
	Mt.Simon	Sandstone		MT.SIMON

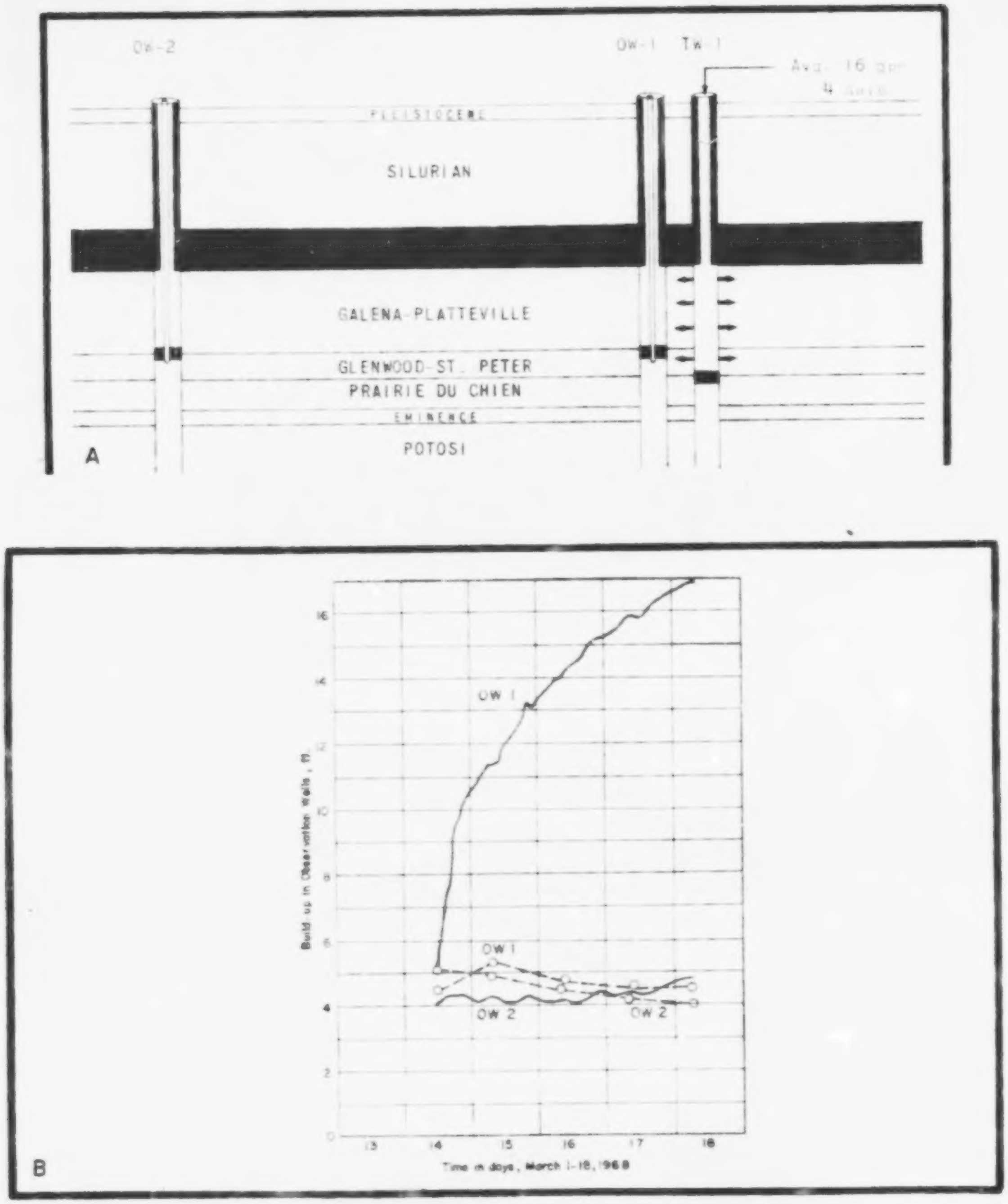
AQUIFER SYSTEMS

Fig. 14



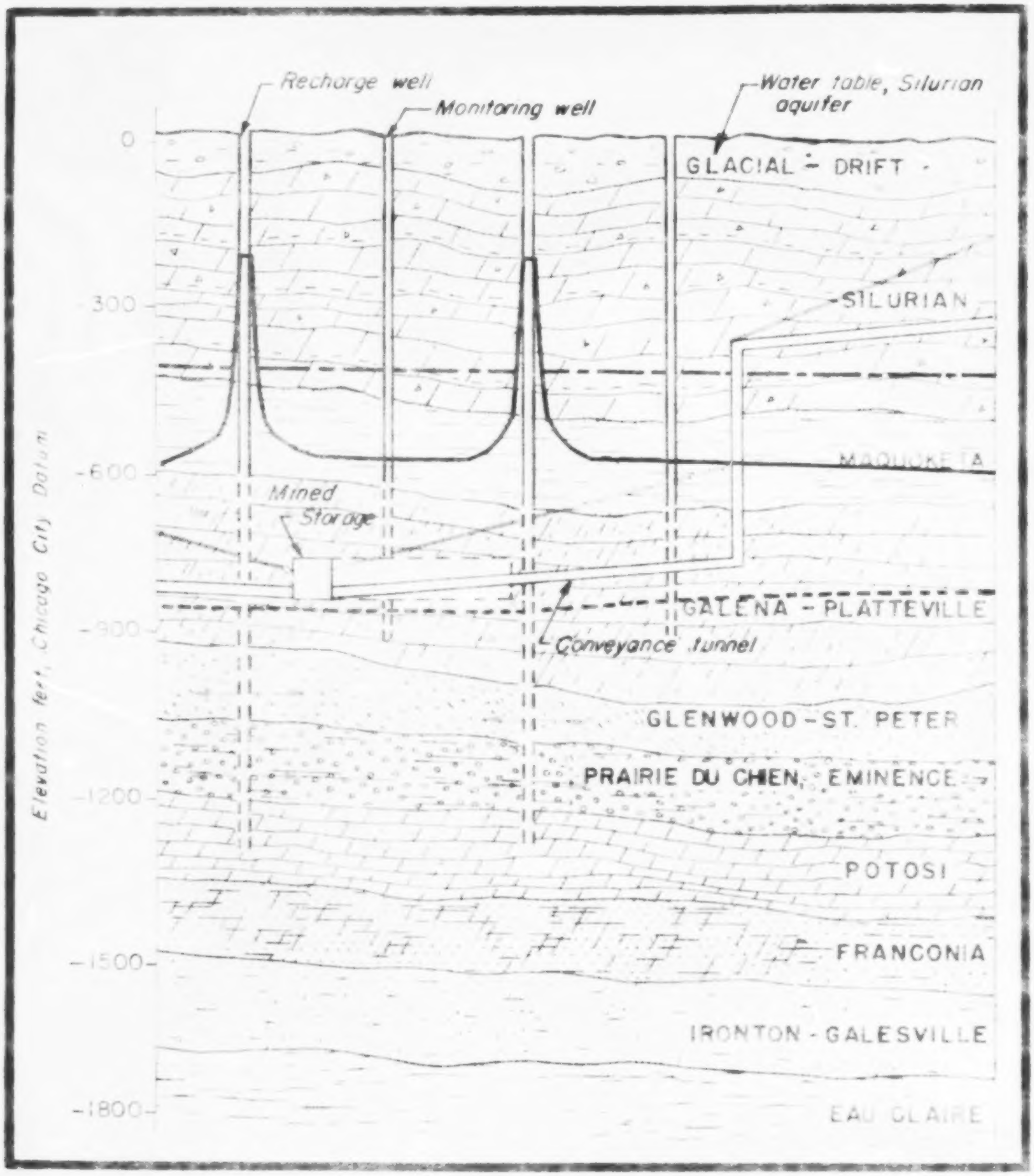
RECHARGE TEST NO.1

Fig. 15



RECHARGE TEST NO. 2

Fig. 16



Recharge well
Monitoring well
Water table, Silurian aquifer
GLACIAL - DRIFT
SILURIAN
MAQUOKETA
Mined Storage
GALENA - PLATTEVILLE
Conveyance tunnel
GLENWOOD-ST. PETER
PRAIRIE DU CHIEN, EMINENCE
POTOSI
FRANCONIA
IRONTON-GALESVILLE
EAU CLAIRE
Elevation feet, Chicago City Datum
0
-300
-600
-900
-1200
-1500
-1800

AQUIFER PROTECTION

Fig. 17

PUMPING TEST INFORMATION – DEEP AQUIFER TEST SITE

Hydrogeologic Unit Tested	Depth (ft.) Top of Bottom Packer	Depth (ft.) Bottom of Top Packer	Diameter (in.)	Penetration (ft.)	Date Tested	Length of Test (min.)	Non-pumping Level (ft.)	Pumping Rate (gpm)	Draw-down (ft.)	Specific Capacity (gpm/ft.)
Silurian Aquifer	495.0[1]	60.0[2]	16	435.0	12/11/67	158	24	40	407.0	0.098
Cambrian-Ordovician Aquifer	1684.0[1]	614.0[2]	12	1070.0	2/14-16/68	1844	434	380	26.4	14.400
Galena-Platteville	900.0	614.0[2]	8	286.0	2/23/68	31	439	40	401.0	0.100
Glenwood-St. Peter	1016.9	919.6	8	97.3	2/19-21/68	1773	435	41	310.1	0.132
Prairie Du Chien, Eminence & Potosi	1366.7	1116.1	12	250.6	2/25-29/68	1825	434	350	26.9	13.020
Franconia	1478.6	1378.7	12	99.9	2/3-5/68	1841	434	300	237.0	1.266
Ironton-Galesville	1684.0[1]	1499.6	12	184.4	1/28-30/68	1808	434	300	100.0	3.000

[1] Bottom of well.

[2] Bottom of casing.

NOTE: TW-1 was the pumped well in all but the Glenwood-St. Peter and Galena-Platteville tests. OW-1 was the pumped well for these two tests.

Table #1

PUMPING TEST INFORMATION – SPECIFIC CAPACITY WELLS

Well No.	Depth (ft.)	Diameter (ft.)	Penetration (ft.)	Date Tested	Length of Test (min.)	Nonpumping Level (ft.)	Pumping Rate (gpm)	Drawdown (ft.)	Specific Capacity (gpm/ft.)
				Silurian Tests					
SW-1	453	12	393	12/6-7/67	49	35	10	121	0.083
SW-2	550	12	475	12/13/67	224	46	30	332	0.090
SW-3	426	12	381	1/8/68	677	16	56	385	0.145
SW-4	549	12	457	1/29-30/68	629[1]	42	60	347	0.173
				1/31-2/1/68	699[2]				
SW-5	489	12	431	1/22-23/68	720	28	60	64	0.938
SW-6	533	12	460	2/15-16/68	721	46	30	357	0.084
				Galena-Platteville Tests					
SW-1	874	8	316	12/6-7/68	720	453	40	145	0.276
SW-2	989	8	294	1/8-9/68	719	357	30	292	0.103
SW-3	913	8	319	1/19-20/68	721	433	50	267	0.187
SW-4	970	8	291	3/5/68	56	395	30	411	0.073
SW-5	882	8	325	1/30-31/68	720	503	60	53	1.132
SW-6	970	8	312	2/28/68	45	431	40	398	0.101

[1] Length of test before acidizing.

[2] Length of test after acidizing.

Table #2

Section 10

THE CONTRACTORS VIEWPOINT OF THE HARD ROCK MECHANICAL MOLE - WHAT'S CAUSING DOWNTIME? WHAT DO THEY WANT?

by

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CONTRACTOR'S VIEWPOINT OF THE HARD ROCK MECHANICAL MOLE
WHAT'S CAUSING DOWN TIME?
WHAT DO THEY WANT?

I believe the tunnel contractor's greatest dream is to bid a tunnel with a mechanical mole, estimating the exact cost and time and when he holes through. The permanent lining, if it requires lining, is within one day of completion.

Digressing from the specifics of the subject title, I would like to approach the problems from the overall concepts of Mechanical Moleing.

The subject of Mechanical Moleing goes far beyond the machine itself. Let us view the subject on the broad concept and break it down into the respective components and analyze each part. Mechanical Moleing can be broken down into four basic categories, namely: (1) determination of ground conditions ahead of moleing; (2) the Mechanical Mole itself; (3) muck removal from the tunnel and transportation of men and supplies; (4) ground support methods (both temporary and permanent).

Up to the present time, I believe that determination of ground conditions ahead of moleing is the one feature that the tunnel contractor needs the most help in to save valuable down time and extremely high costs. In so many cases if the contractor knew the conditions ahead of moleing, he could take care of the problem prior to the element of surprise. These conditions could be fault zones, running ground, water, gas, squeezing or heaving ground, just to name a few of the surprises mother nature has in store for the underground excavator.

I would like to show a few examples of ground conditions that were not anticipated in a moled tunnel by prior exploration. The first figures are of the Oso Tunnel, a part of the San Juan Chama Project in southeast Colorado.

Fig. 1 shows a tunnel profile with the bad ground conditions shown in a circle. There were nine holes drilled (down to tunnel grade) along the tunnel center line prior to bidding. A contractor who bid on the job drilled an additional hole in the valley at Sta. 660+50 to check for a possible fault. There was not a fault and the formation conformed in dip and strike of the shale bedding with all the other holes.

The mole was started at the down stream portal and moled into about Sta. 721+50 when water, sand, gravel was encountered and caused the back of the tunnel to cave in on the head of the mole without any warning. The tunnel was then enlarged conventionally to allow for steel spiling to be driven over the mole. The mole was then taken out of the tunnel to give room for driving conventionally through the bad ground. As soon as the mole was out of the tunnel, three diamond drill holes were drilled in the face of the tunnel to try to determine the ground con-

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ditions. The holes were drilled 150 ft. long, +15^o, - 15^o, and level and it was found that silt, sand, gravel, boulders and water were directly ahead and the conditions bad enough so it was almost impossible to drill any further than the 150 ft. At this point seven holes were drilled from the surface and it was determined that there was 1000 ft. of glacial till to drive through before reaching the shale as originally expected.

(Fig. 2) In this case there was a hole at the down stream portal and one 3000 ft. from the portal showing the same dip and strike of the shale bedding and yet there was 1000 ft. of extremely bad ground in between which could not be determined by the drill holes or surface examination. If a good seismograph program had been performed prior to bidding, this condition would have been determined and remedial action could have been taken. The ground could have been grouted or drained of water and while this was going on the contractor could have been driving the tunnel from the other portal. The 1000 ft. of bad ground tunnel could have been driven for $300 to $400/foot of tunnel instead of the $1000/foot.

Fig. 3 shows the extent of the steel spiling to support the ground prior to removing the mole. Fig. 4 shows the 6 in. channel spiling as driven so as to support the tunnel. Fig. 5 shows the face of the tunnel being breast-boarded to keep the face from running prior to and during spiling.

The second case of surprised ground conditions was at the water-hollow tunnel job, a part of the Central Utah Project at Strawberry, Utah. There were very few holes drilled on this project and only short holes close to the both portals as shown in Fig. 6. There were surface outcrops of the general formations and from a stratographic standpoint you could get a good idea of what ground conditions you might expect.

There were two potential fault zones that showed up from surface examinations. In the driving of the tunnel, proper precautions were taken by putting in steel sets and installing water pumping lines to take care of the water. The tunnel was driven through these areas with little or no trouble.

At a point about 3 mi. in from the downstream portal, the tunnel went from good ground into an extremely bad fault zone carrying gravel, fault-gauge and water up to 1200 gal./min. at very high pressures. The length of this fault zone was 350 ft. long and changed rapidly from one type of disturbed ground into another, gouge to conglomerate, to gravel to gouge to siltstone, etc. It was necessary to spile more than one-half the distance solid with 48# rail, back packed with hay and timber to prevent the ground from running and also for support of the heavy tunnel arch. The steel channel spiling was driven by means of pneumatic spaders. The large 48# rail was driven with the gripper pads of the mole itself, this system was very fast and efficient.

Fig. 7 shows the channel spiling on the tunnel ribs with wooden blocking to prevent the ground from running. Fig. 8 shows the 48# rail in the back of the tunnel with the hay between rails. Fig. 9 shows the mole thrust cylinder covered with sand, gravel, gouge prior to cleaning up with vacuum car. An extension pipe of 150 ft. long was put on the vacuum car and sucked the loose material from around the mole and up to the tunnel face. Fig. 10 shows the vacuum car.

The third case of unexpected ground conditions at the Azotea tunnel which is a thirteen-mile tunnel part of the San Juan Chama Project in northern New Mexico. The problem in this tunnel was in a 8000 ft. stretch in the center of the tunnel. The problem was not a major fault itself, but the side effects of a major fault. There were a series of displacement or drag effects that appeared tight and undisturbed at the time of mole penetration but started to take weight and movement occured many months later. If this condition had been known prior to moleing, then the tunnel could have been bored large enough to put in support steel. A geophysicist in the area had made determinations from oil well drill logs close to the tunnel area prior to the driving of the tunnel that showed these conditions would exist but he could not be heard by the proper people and gave up. His maps made prior to tunnel driving showed almost exactly the problem area.

I believe all the stated examples could have been determined ahead of time with adequate geological and geophysical examination and determination. If the conditions were determined ahead of moleing the contractor would have been better prepared to cope with the problems, saving considerable time and money. He could have had the proper tools and supplies on the job for grouting, ground support, water handling, or any other proper construction procedures, etc. I believe the oil field geophysicists with their exotic means of ground determination in conjunction with our tunnel geologists can aid and assist the excavator in the area of ground determination ahead of moleing. At the present time I am informed there are studies going on in California with "sonic" methods being used in an active tunnel, to determine ground conditions ahead of excavation and a report is to be published early in the coming year by the University of California.

I would like to now pass to the third phase of our tunnel breakdown, namely muck removal and transportation. Muck removal in tunnel moleing shows the greatest overall delay time. In the case of the Oso tunnel where the contractor made as high as 412 ft. in 24 hr., the mole availability was only 68% due to muck removal delays in train switching, etc. Muck removal means, such as slurry, hydraulic or pneumatic systems, show the greatest promise for continuous, fast, efficient muck transportation from the tunnel excavation. In conjunction with such systems, means must be developed for the transportation of men and supplies into and out of the tunnel heading. The method of transportation for men and supplies would be directly related to the muck removal system, but could be by rail, off-track equipment or mono-rail.

The fourth phase in tunnel moleing, namely, ground support temporary and permanent, is directly related to the other three phases of tunnel moleing. Ground conditions tell us what type of supports will be needed and how and when. The moleing rate tells us how and when, and the muck removal system tells us how and when. For example, if the ground is weak and not self-supporting, then support must be put in around the mole at the immediate tunnel heading. If the ground is self-supporting then support, if required, can be put in back of the congestion of the moleing operation. If a muck removal system is used other than by rail transportation, then the permanent lining can be put in with ease in conjunction with the moleing progress. The installation of permanent lining of the tunnel in conjunction with the moleing operations approaches the ultimate in tunnel driving, where permanent lining is required. The system was tried at the Azotea tunnel in conjunction with the slurry system of muck removal. It was used successfully for 1000 ft. of tunnel but had to be abandoned because of continual failures of the slurry system. The tunnel was then driven with a Robbins Mole, muck cars and temporary support until the tunnel was holed through and the permanent lining was installed. (The present methods most generally used for ground support at the tunnel face are circular steel ribs with metal or wooden lagging and roof bolts with plates, steel lagging or wire mesh.)

I believe a continuous method of applying gunite or shotcrete at the molehead should be developed. This could be done on a rotating arm the proper distance from the tunnel rib and advance controlled with the advance of the mole.

There is also a patent pending for a continuous rolled steel lining similar to the method of rolling ventilation pipe, this could be put in at the molehead on a continuous basis.

Now let's consider the second phase of Mechanical Moleing, the mole itself. Contrary to some people's thinking, I believe of the four phases of Mechanical Moleing, the moleing phase is presently far ahead of the other three. However, there are many improvements to be made in present day moles, and I would like to present what I think should be developed.

There should be two sets of gripper pads, one set in the front and one set in the rear. This would allow a choice of positions for gripping in case the ground was bad in front and needed support or could not properly grip in bad ground. This would also assist in case the mole was required to go on a plus or minus grade. A vertical raising or lowering on the gripper pad support could be developed to readily give alignment for grade control, this would remove a control pad from the invert of the tunnel to allow greater working room. Two sets of gripper pads would eliminate intermittent moleing progress because of regripping time. Differential and controlled gripping pressures on either side of the tunnel gripper pads could be advantageous in variable ground conditions and also could aid in guidance control. Differential gripping pressures could be advantageous in enlarging the tunnel section to allow for passing tracks, switches, or transformer stations, etc., etc.

The main support beam of the mole should be as small as possible and high off the invert as is possible to give greater working room on the tunnel invert.

Moles should be made to be able to be knocked down into small component parts so as to be able to be moved into and out of working faces with ease and speed. The access to the molehead should be such that it is possible to make rapid changes of cutter bits.

Lubrication systems should be made automatic or semiautomatic so servicing can go without mole stoppage. Automatic controls should be put on moles, not necessarily to eliminate manpower, but to make for a smoother operation for alignment control and steady penetration rates. Rapid changes in line and grade are hard on cutter bits, the molehead, the thrust bearing and the main frame itself.

It would be very advantageous to have the molehead so it could be easily expanded from the operator's control cab for tunnel enlargement, this was used in some moles with fair success but needs further development. Moles should have a longitudinal center opening so that test drilling can go on for ground determination independent of the progress of the moleing operation.

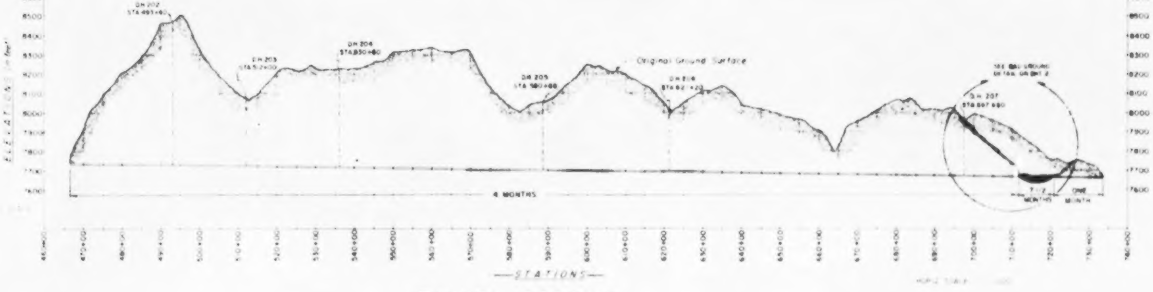
It would be advantageous for moles to have a variable speed head rotation for maximum efficiency for ground penetration in variable ground conditions. (This lesson can be learned from our oil drilling friends.)

Moles should be equipped with segmented shields for ground support until temporary support can be put in. Each segment of the shield should be independently operated, both in and out on radius and back and forth along the tunnel line. This would allow the installation of support in extremely bad ground.

Moles should be equipped with vacuum machines to clean the tunnel invert to allow the pouring of sub-invert or final invert directly behind the mole.

In conclusion I would like to state the following. I believe there should be a standardization in tunnel sizes within certain limitations. It seems to me we are defeating the purpose of rapid excavation to have to spend such large sums of money for a system to have it outmoded by a size change. I would suggest, for example, that with very few exceptions all tunnels could be standardized into three sizes: 12 ft., 18 ft. and 28 ft., each one of which would have a plus or minus one-foot variance. This would allow a contractor a better opportunity to develop and amortize a rapid excavation system over many more feet of tunnel to the overall financial benefit to the owner.

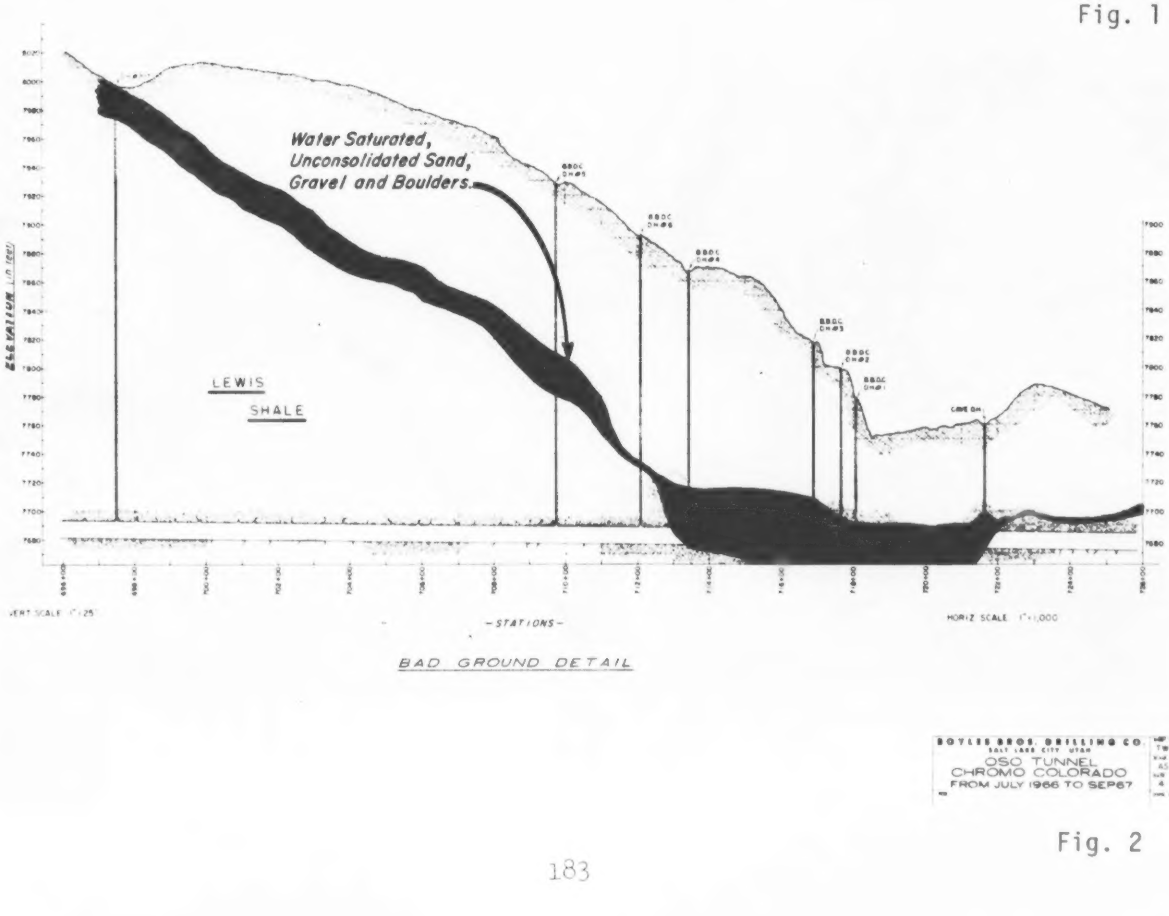
I believe with some of the aspects I have mentioned, downtime can be cut to a minimum and that these are some of the things a tunnel contractor wants and needs.


ELEVATIONS (in feet)
8600
8500
8400
8300
8200
8100
8000
7900
7800
7700
7600
DH 202
STA 493+60
DH 203
STA 512+00
DH 204
STA 530+60
DH 205
STA 590+88
Original Ground Surface
DH 206
STA 621+20
DH 207
STA 697+80
4 MONTHS
7 1/2
MONTHS
ONE
MONTH
470+00
480+00
490+00
500+00
510+00
520+00
530+00
540+00
550+00
560+00
570+00
580+00
590+00
600+00
610+00
620+00
630+00
640+00
650+00
660+00
670+00
680+00
690+00
700+00
710+00
720+00
730+00
740+00
—STATIONS—
HORIZ SCALE

TUNNEL PROFILE

BOYLES BROS. DRILLING CO.
SALT LAKE CITY UTAH
OSO TUNNEL
CHROMO COLORADO
FROM JULY 1966 TO SEP67
ONE (1)
AS SHOWN
4 9 70

Fig. 1


ELEVATION (in feet)
8020
8000
7980
7960
7940
7920
7900
7880
7860
7840
7820
7800
7780
7760
7740
7720
7700
7680
Water Saturated,
Unconsolidated Sand,
Gravel and Boulders.
LEWIS
SHALE
BBDC
DH#5
BBDC
DH#6
BBDC
DH#4
BBDC
DH#3
BBDC
DH#2
BBDC
DH#1
CAVE DH
696+00
698+00
700+00
702+00
704+00
706+00
708+00
710+00
712+00
714+00
716+00
718+00
720+00
722+00
724+00
726+00
VERT SCALE 1"=25'
-STATIONS-
HORIZ SCALE 1"=1,000

BAD GROUND DETAIL

BOYLES BROS. DRILLING CO.
SALT LAKE CITY UTAH
OSO TUNNEL
CHROMO COLORADO
FROM JULY 1966 TO SEP67
TWO (2)
AS SHOWN
4 14 70

Fig. 2

Fig. 3

Reproduced from best available copy.

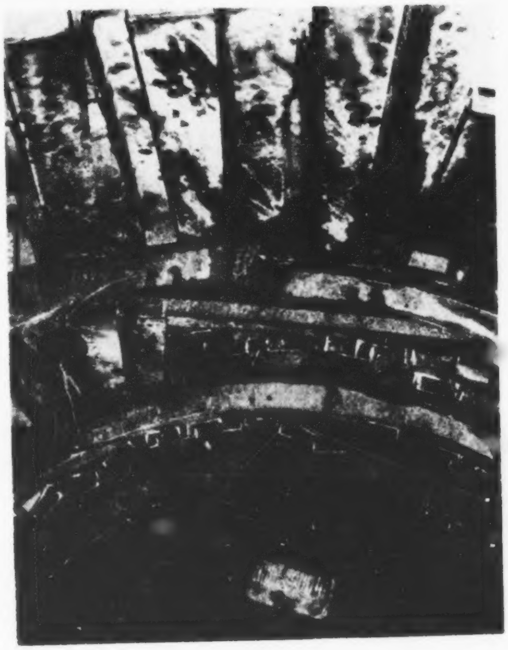


Fig. 4

Fig. 5

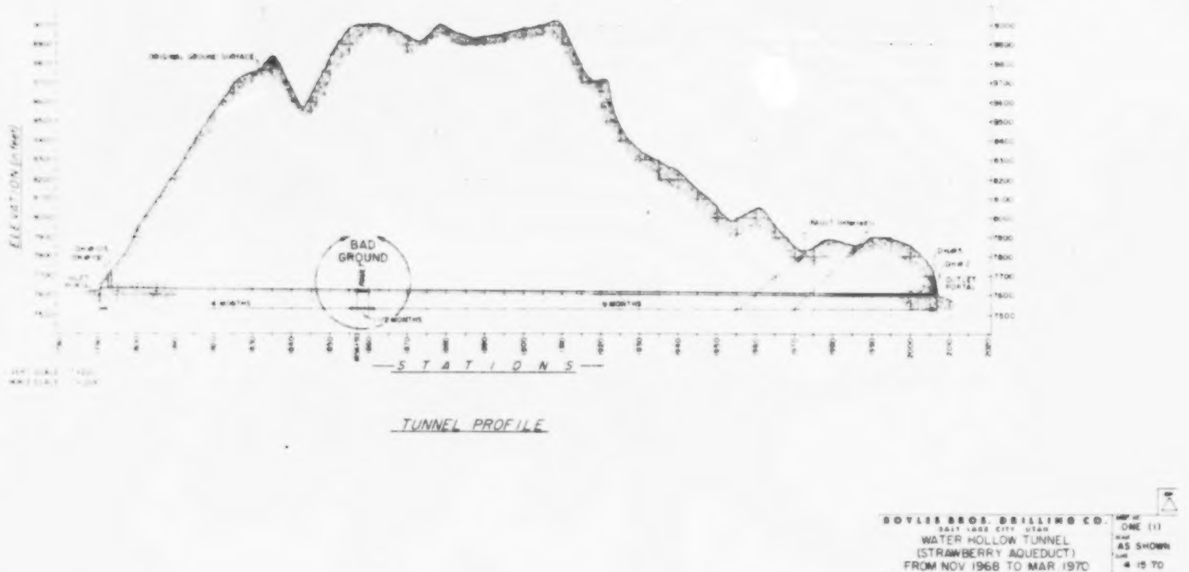

ORIGINAL GROUND SURFACE
ELEVATION (in feet)
BAD GROUND
2 MONTHS
STATIONS
TUNNEL PROFILE
OUTLET PORTAL
9000
7500
BOYLES BROS. DRILLING CO.
SALT LAKE CITY UTAH
WATER HOLLOW TUNNEL
(STRAWBERRY AQUEDUCT)
FROM NOV 1968 TO MAR 1970
ONE (1)
AS SHOWN
4 15 70

Fig. 6

Reproduced from best available copy.

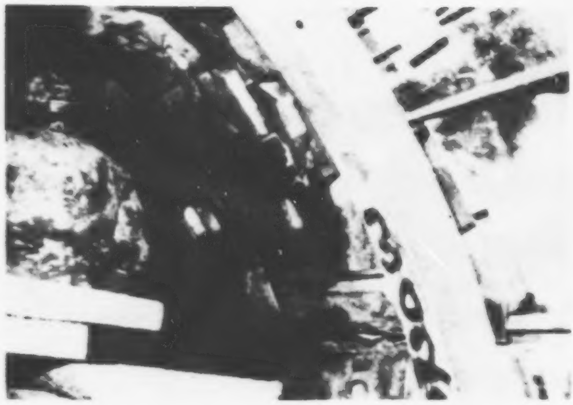


Fig. 7

Fig. 8

Fig. 9

Reproduced from best available copy.

Fig. 10

Section 11

RAPID EXCAVATION IN HARD ROCK:
A STATE-OF-THE-ART REPORT

by

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RAPID EXCAVATION IN HARD ROCK--
A STATE-OF-THE-ART REPORT

ABSTRACT

In the United States, 12 tunnels have been machine bored since 1955 in rocks of over 20,000 psi compressive strength. One-half of these operations were performed in a manner superior, in both the speed of boring and the quality of the opening, to conventional drill and blast methods. The other one-half were either partially successful or failures, and the machine had to be replaced by conventional methods. Boring machines have been making steady improvement in their ability to bore hard rock, and some recent tunnels have been successfully bored in rocks of up to 30,000 psi compressive strength with maximum advance rates of up to 1,500 ft./mo.

This paper describes the evolution of present day tunnel boring techniques. Emphasis is directed toward selected cases from the past decade which are discussed in some detail. The data presented has, in many cases, been generated by Bureau of Mines personnel during on-site studies of the particular job. Wherever feasible, samples of the rocks being bored were returned to the Twin Cities Mining Research Center for determination of the physical properties.

This paper presents the significant problems and accomplishments for various actual operations. Wherever possible, it presents physical characteristics of the rock encountered to aid the audience in evaluating rock hardness.

Trends for the future are forecast relying on objectives as developed by the OECD[1] as well as on experience of Bureau of Mines personnel who for years have followed developments in rapid excavation[2] technology.

INTRODUCTION

Tunneling by machine is generally classified as hard rock or soft ground tunneling. Each classification has unique problems and requires different techniques and equipment. This paper describes the state of the art of rapid excavation in hard rock.

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[1] Organization for Economic Cooperation and Development Advisory Conference on Tunneling; Washington, D.C., June 22-26, 1970.

[2] The term "rapid excavation" is defined in this paper as excavation performed by tunnel boring machines or moles and does not include conventional drill and blast methods.

To begin the discussion of machine tunneling in hard rock we must first define the term "hard rock." This definition must necessarily be arbitrary since the ability to fragment rock is a function of both time and the fragmentation process. For example, what is considered hard rock for today's machines may be considered soft rock in the future; and rock which is impossible to break by conventional moles may be readily broken by hydraulic processes.

Considering only present-day machines, however, we would define hard rocks as sediments or metasediments (metamorphosed sedimentary rocks) with a uniaxial compressive strength greater than 20,000 psi. This would include the common varieties of dolomite, limestone, sandstone, shale, marble, slate, etc. In the definition of hard rock, we would include other metamorphics and igneous rocks with uniaxial compressive strength greater than 10,000 psi[3]. This classification would include rocks such as schist, granite and basalt. In addition, we would include other difficult to mole rocks such as those that are blocky by nature, badly fractured in situ, or conglomeritic in nature. Soft rock is alternatively defined as sediments or metasediments with uniaxial compressive strengths less than 20,000 psi and as other metamorphics and igneous rocks of less than 10,000 psi compressive strength.

DEEP-TUNNEL PROJECTS

Deep-tunnel projects, such as those being developed in the Metropolitan Sanitary District of Greater Chicago (MSDGC), offer many benefits to our environment. The combined storm and sewer system includes an interceptor network to alleviate flooding of specific urban areas. The combined system offers economies because it eliminates construction of two separate systems. A combined system may cost only one-third as much as a separated system, and the incorporation of hydroelectric pumped storage may contribute income which can reduce the net cost of the entire system.

Additional benefits are realized by a combined storm water and raw-sewage system. The combined sewer system is designed to deliver effluent of a quality which will meet new stringent standards. The storage capacity is a safety valve to elminate problems of sewage overflow during periods of high rainfall.

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[3] Note that while the definition of rock hardness was based entirely on rock compressive strength, it is well recognized that this parameter is not a complete indicator of boreability. There are many researchers, including those at the Twin Cities Mining Research Center, who are trying to develop an accurate, universally accepted boreability index. Until one is developed, however, rock compressive strength will remain as the one common parameter recognized by the entire tunneling community as being related to boreability.

The smoothing out of the power supply-demand curve by the use of pumped storage offers advantages. Because power can be generated to meet high demands using release of surface water to underground storage, customer needs may thus be met without the atmospheric pollution associated with some coal burning plants.

Finally, if water table recharge by injection wells is incorporated, ground water levels, which historically have been falling, could be maintained at desirable levels. All these advantages are in line with the desperate need of a society intent on improving its environment.

Public officials equating the advantages of environmental and safety improvements with fiscal and budgetary constraints are necessarily sensitive to cost factors including excavation costs.

Rapid excavation in hard rock holds promise for diminishing project costs; however, such excavation is sensitive to the laws of supply and demand. We are all aware of the pace of excavation needed to provide efficient services demanded by the public. Advances in fragmenting hard rock are needed now to provide high-speed underground transportation, efficient underground emplacement of utilities, and economical development and production of the Nation's mineral reserves. Our future needs will be even more demanding.

TUNNEL BORING - PAST AND PRESENT

Although tunnel boring machines have evolved over the last century, the 1950's saw the first extensive use of mechanical moles.

The early successes were achieved with Robbins boring machines in the tunnels of the Missouri River Basin in the 1950's. The rocks encountered, although bored successfully, might well be considered to be very soft sedimentary rocks, probably in the 5,000-to 10,000-psi range.

The dawn of present-day tunnel boring probably occured in the 1960's. During the sixties, noteworthy advances were made, but not without equally spectacular failures.

Before beginning a discussion of tunnel boring in hard rocks, we will summarize the results achieved during the sixties in what we have defined as soft rocks. Although this brief discussion will make no mention of earth-boring ventures, it should be remembered that many earth-boring jobs have been undertaken during the past two decades.

In 1961, a Robbins machine operated with limited success in the Kerr-McGee Corp. Section 33 mine near Grants, New Mexico, in sandstone ranging from 1,200-to 2,500-psi. Here progress was limited both by adverse water conditions and by rock too soft to withstand forces required to anchor the mole. In 1961, a Robbins machine bored 8,000-to 15,000-psi material in the Homer-Wauseca mine in Upper Michigan with limited success, primarily because of directional control problems and adverse water conditions. Another project in 1963 involved two Hughes machines and one Robbins machine boring an Arizona water-diversion tunnel for the Phelps Dodge Corp. Success on this particular job was also limited by water problems as well as guidance problems. The material involved was sandstone ranging from 2,000-to 15,000-psi. Guidance problems on many tunnel boring jobs led to introduction of the laser for improved directional control of boring machines in the mid-sixties. In 1965, in the Navajo No. 1 tunnel in New Mexico, the Hughes Betti I machine, using a sophisticated laser guidance system, successfully bored a tunnel in 5,000-to 10,000-psi sandstone and deviated less than 5/8 in. along the entire tunnel. Starting in 1965, Jarva machines operating in St. Louis successfully bored sewer tunnel in limestone ranging in compressive strength from about 15,000-to 20,000-psi. At approximately the same time, 1965 through 1967, Robbins moles successfully bored tunnels in shales with strengths up to 10,000 psi at the San Juan-Chama project in northern New Mexico and southern Colorado. The San Juan-Chama project, consisting of three tunnels, set a record by boring about 420 ft. during one 24-hr. period. This spectacular achievement was made in the Oso tunnel which was also the site of a rather classical case of changed conditions. Prospect holes spaced at about 1,000-ft. intervals failed to delineate a zone of about 900 ft. of loosely cemented conglomerate which forced a temporary cessation of machine tunneling.

In 1967, a Calweld boring machine successfully bored a storm sewer in the St. Peter sandstone underlying Minneapolis. The sandstone has a uniaxial compressive strength less than 500 psi. Although the machine performed successfully, an inrush of water occured shortly after boring was completed, with the result that the machine was buried in debris for several months.

So far, we have mentioned but a few tunnel boring operations carried out during the past decade. We have reserved discussion of those operations which took place in what we have defined as "hard rock." These cases are reserved for the main topic of discussion of this paper.

THE STATE OF THE ART OF HARD ROCK BORING

A typical U.S. manufactured hard rock mole can be described as a self-advancing rotary drilling machine which cuts the full face of the tunnel in a semicontinuous fashion. Most machines consist of an inner and an outer frame. The inner frame of the mole usually carries the cutterhead which both rotates and advances forward as drilling proceeds. The outer frame is kept stationary during the cutting process by means of large hydraulic jacks which are forced out against the tunnel walls.

From this stable position, the thrust and torque of the cutterhead are reacted. The cutterhead is fitted with a number of individual cutters which cut or spall the rock as the cutterhead is rotated and forced against the tunnel face. The cuttings are collected by buckets behind the cutterhead and are discharged onto a conveyor which carries them to the muck haulage system behind the machine. To illustrate some of these features a Jarva Mole is shown in Figure 1.

BASIC CYCLE

The basic operating cycle (Figure 2) of a typical boring machine is as follows: 1) To begin the stroke, the mole is aligned in the desired direction of advance by the rib jacks which shift the axis of the mole in the desired direction. These rib jacks also serve to lock the machine securely in the tunnel when boring. 2) Boring proceeds as the thrust cylinders force the rotating cutterhead into the tunnel face. The torque and thrust of the cutterhead are resisted through the mole by the rib jacks. 3) At the end of the forward stroke of the thrust cylinder, the rib jacks are released, the support jacks are lowered, and the machine is moved ahead by retracting the thrust cylinder. 4) The mole is again aligned in the tunnel and is ready to bore another stroke.

BASIC OPERATIONS IN HARD ROCK TUNNEL BORING

Rock Disintegration

The rock disintegration system of a tunnel boring machine is composed of the cutterhead and the individual cutters mounted upon it. All hard rock machines manufactured in the U.S. use some type of rolling cutters which are usually hard-faced at selected spots with tungsten carbide or which have sintered tungsten carbide inserts on the cutting surfaces.

The most popular cutters are either single or multiple disks with tungsten carbide inserts on the periphery, or roller-shaped button bits which have tungsten carbide inserts mounted around the entire surface (Figure 3). The button bit is used in the hardest rocks where the rock will not break out readily between two adjacent kerfs. The single or multiple disks are used in hard rock where chipping between adjacent disks is possible and thus give a larger chip product and usually faster penetration. Most cutters have replaceable cutter shells and bearings so that, depending on which fails first, either the bearings or cutter surfaces can be replaced separately.

Depending on the type of machine, the center section of the main cutterhead can revolve either with the main cutterhead or independently. If the center of the cutterhead revolves independently, it is usually equipped with a tricone type cutter and rotated at a speed of 30 to 60 rpm. Usual practice is to use a thrust of 50,000 lb./ft. of cutterhead diameter and to use a rotary speed equal to 90 divided by the diameter in feet. The larger the cutterhead diameter, therefore, the slower it is revolved; the speed of the outside cutters is thus held at an acceptable level. The majority of the hard rock moles described in this report had a rotary speed of 8 to 9 rpm, a maximum thrust capability of between 1 and 1-1/2 million lb., and a maximum torque capability of between 300,000 and 500,000 ft. lb.

Materials Handling

After the rock is broken from the tunnel face, it falls to the invert of the tunnel where it is removed by buckets mounted behind the cutterhead. As the cutterhead rotates, these buckets scoop up the broken material from the invert and deposit it on the machine conveyor. This is a belt conveyor, usually 24 in. wide, which carries the muck to the end of the machine and, in the majority of cases studied, onto a trailing conveyor. The trailing conveyor, which advances along with the mole, is 20 to 30 in. wide and 200 to 300 ft. long and carries the muck from the mole to the muck cars. Except for the White Pine machine, which used conveyor haulage, all the machines studied in this report used train haulage. The locomotive type and size varied as did the muck cars. Most operations used a California switch to pass the empty and loaded trains in the tunnel. The muck trains on the return trip generally carry supplies, such as cutters or support materials, back into the tunnel.

Primary Support

In hard rock boring, where the rock is not excessively fractured, primary tunnel support requirements are usually minimal. The very nature of machine boring is such as to create a minimum disturbance to rock outside the tunnel walls. The result is a stable opening which requires fewer and lighter supports. Of the tunnels studied in this report, over one-half required little or no support. Except for one tunnel in blocky ground, the rest required only minor roof bolting or shotcreting to control the tunnel roof. In two of the MSDGC sewer tunnels in Chicago (Jobs 8 and 9) the bored tunnel surfaces were of such high quality that the final concrete lining was eliminated. Blocky ground and occasional rock falls or water inflows from fault zones, solution cavities, etc., still present problems in hard rock tunnels. Therefore, many hard rock machines have both partial shields around the top of the machine to protect them from these hazards and roof pinning drills for the installation of roof bolts and mats for temporary support. All machines can be equipped with mechanical aids for setting ring beam supports if greater support is necessary.

Survey and Control

The maintenance of line and grade has become routine in recent years with the use of the low power, continuous output laser which can project a thin beam of visible light for long distances. In a typical installation the laser is set up on the tunnel rib some hundreds of feet behind the tunneler by conventional surveying methods. A reference beam on the correct line and grade is then projected toward the mole where it strikes two targets mounted at the front and rear of the machine. To keep the mole on course, the machine is shifted to keep the beam centered on these targets. In recent years many of these targets have been built with photoelectric cells which are activated when hit by the laser beam. These photoelectric cells provide a continuous readout of the mole's attitude in the tunnel and can also be used to signal a servocontrol system to automatically adjust the mole to keep it on proper course. Most of the newer tunnel boring machines studied in this report used laser guidance systems.

Environmental Control

Dust generation has been one of the most serious environmental problems associated with tunnel boring machines. This problem has been most successfully solved by two methods, used either singly or in combination. One method is to use water sprays near the cutters to reduce the airborne dust. Controlling the water to these sprays is critical, however, as too much water will make a slurry of the cuttings while too little causes the cuttings to become too sticky to handle. The other method is to isolate the cutterhead with a flexible shroud and evacuate the dust-laden air from this area with a vacuum system. This air can then be run through a wet scrubber or exhausted directly from the tunnel.

CASE HISTORIES

Using our definition of hard rock, the entire U.S. experience in this field reduces to 12 jobs. Approximately one-fourth of these jobs were definitely not successful; i.e., the machines could not bore the rock and had to be removed. Another one-fourth were partially successful; i.e., they could bore the rock, but very slowly. The other one-half were considered very successful; i.e., the penetration rate, quality of the opening, and sometimes the cost, were superior to what could have been accomplished by conventional methods.

The jobs we will be describing are presented in chronological order, with the earliest being discussed first. Since most of these had similar muck handling, guidance, and environmental control systems, these will not be discussed in detail. A summary of all pertinent tunnel data and boring machine data for each job is given in tables 1 and 2 at the end of this report.

The four companies now manufacturing hard rock tunnel-boring machines in the U.S. are Jarva, Lawrence, Robbins and Calweld. Each of these companies has manufactured successful hard rock machines. Other U.S. firms manufacturing tunnel-boring machines, but whose machines have not been used in what we have defined as hard rock, are the Hughes Tool Company the Mining Equipment Manufacturing Company (MEMCO) and Dresser OME.

The first hard rock tunnel bored in the U.S., by our definition, was in Chicago, in 1956. The project was a 9-ft.-diam sewer tunnel constructed in limestone with a compressive strength of 18,000-to 25,000-psi. Although information on this job is scarce, we assume that the rock was the same kind as that in which the present day Chicago sewers are being constructed.

The mole used on this job was a Robbins model 103 and the fifth one built by that company. At 17 tons, this was a light machine by today's standards. The cutterhead was fitted with both disk and drag cutters. This machine had a thrust of only 110,000 lb. but had a torque of 138,000 ft. lb. This high torque to thrust ratio was probably made necessary by the drag cutters which require a large tangential force to move them across a rock surface.

Although the mole achieved an average penetration rate of from 2 to 4 fph, it was not successful because the drag cutters were unsuited to the hard limestone. The carbide inserts in these cutters were hard enough to cut the rock, but the bits suffered from excessive shock loading and the inserts could not be kept in their tools. The replacement of these inserts was a major source of downtime and expense (9).[4]

Experience gained on this job led to the successful boring of a similar sewer tunnel in Toronto, Canada, the next year. This mole, also a Robbins machine, was modified to increase the structural strength, as well as the torque and thrust capabilities. Probably most important, the drag cutters were replaced with rolling disk cutters. This machine achieved advance rates of over 100 fpd.

Job No. 2

The next hard rock tunnel was the Richmond water tunnel begun in 1964. This 12-ft.-diam., 5-mi.-long tunnel was designed to bring 300 million gallons of water daily from Brooklyn to Staten Island.

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[4] Underlined numbers in parentheses refer to itmes in the list of references at the end of this report.

The rock bored was the Manhattan schist which is composed primarily of quartz with small amounts of feldspar, garnet and muscovite. With a compressive strength of 25,000 psi, this was the hardest rock attempted by a tunnel borer up to that time.

The mole used on this job was the prototype Alkirk Hard Rock Tunneler, model HRT-12, built by Lawrence Manufacturing Co. (Figure 4). This mole uses the pilot-pull principle. In operation a 17-1/2-in-diam pilot hole is bored some 9 ft. ahead of the main cutterhead. A gripper and rubber packer are then expanded to anchor the pilot drill assembly in the hole, and the hydraulic unit pulls the cutterhead against the face to provide a portion of the total thrust generated by the machine. The balance of the thrust is generated by a conventional rib jack and thrust cylinder arrangement. This pilot drill, in addition to providing thrust, also serves to keep the mole on course and provides additional stability while boring. The main cutterhead was dressed with 53 tungsten carbide button bits and revolved at 9 rpm. The machine had a thrust of 1 millon lb. and a torque of 250,000 ft. lb. The estimated cost of this machine was $500,000, but it was expected to pay for itself from the reduction in concrete used in the lining because of reduced overbreak.

Although the mole achieved an average penetration rate of 4 fph when in operation, it was hampered by machine as well as geological problems for most of the year and required continual adjustment. Just as the machine was becoming debugged, the cutters failed to perform satisfactorily. Finally, after one year of effort had achieved only 200 ft. of tunnel, the mole was pulled off the job and the tunnel was finished by conventional drill and blast methods.

Despite these difficulties, this machine showed that rock as hard as 25,000 psi could be bored at satisfactory rates of advance with a mechanical mole.

Job. No. 3

The first hard rock boring success in the U.S. took place at the Republic Steel Corp. Adirondack Mine at Mineville, N.Y., in April 1967. The project was a 10-ft.-diam. inclined shaft, 765 ft. long and with a slope of 27 degrees.

The rocks encountered on this bore ranged from magnetite ore with a compressive strength of 10,000 psi through horneblende, biotite gneiss, and gray granite gneiss. This last rock has a compressive strength of 35,000 psi.

The mole used was a Jarva Mark 11 equipped with tungsten carbide button and kerf cutters (Figure 5). The penetration rate ranged from 1 to 4 fph and averaged 1.67 fph.

Although the penetration rate was low and the length of the bore was short, this was the first success in hard rock boring in the U. S. The other noteworthy feature of this job was the ability of the machine to bore on a 27-degree downgrade

Job No. 4

Job No. 4, the Lawrence Avenue Sewer tunnel (Figure. 6), is being bored in Chicago, Ill. This job began in March 1968 and as of September 1970, 6,760 ft. of 13-ft 8-in.-diam. tunnel and 3,968 ft. of 13-ft. 4-in.-diam. tunnel had been bored.

The rock being bored is the Niagaran dolomite, a massive, competent, and dry rock. Our tests at the TCMRC show this rock to have a compressive strength of from 17,000-to 32,000-psi with an average of 27,000 psi.

The first 7,000 ft. of this tunnel was bored with the prototype Alkirk Miner, the same as was used in the previously mentioned Richmond Water Tunnel. At the beninning of this project, a number of difficulties were experienced with this machine including an electrical fire. During the last 4 months the machine averaged an impressive 1,000 ft./month.

In December 1969, a new Lawrence machine was installed, and by July 1970 it had completed 4,000 ft. of tunnel known as the Harding Avenue Section. The machine is now working on the remaining 12,670 ft. of the Lawrence Avenue job.

This second mole has achieved some excellent boring rates, as have the other two moles working in the Niagaran dolomite (Jobs 8 and 9). Thus, where conditions are right, even in rock which is quite hard, the excavation rate of tunnel moles has shown steady improvement and holds even greater potential for the future, primarily because of its continuous fragmentation system.

Job No. 5

One of the most successful machine-bored hard rock tunnels began in September 1968 and was completed 9 months later. This was the 20,000 ft. long River Mountains tunnel driven by Utah Construction and Mining Co. with a Jarva mole. This 12-ft.-diam tunnel through the River Mountains was part of the Southern Nevada Water Project, which is designed to bring water from Lake Mead to Las Vegas and other southern Nevada cities.

The rocks encountered along this bore were primarily extrusive volcanic rocks of an extremely complex and variable geology, mainly tuffs and breccias, rhyolite, and rhyodacite. The rhyolites had compressive strengths of from 3,000-to 10,000-psi and the rhyodacites from 4,000-to 23,000-psi. Over 40 fault zones were crossed during this bore and, except for some blocky ground at these faults, they did not present any great problems. Furthermore, the tunnel was dry along its entire length. The combination of fairly soft but competent rock and dry conditions offered nearly perfect conditions for a tunnel boring machine.

The mole used on this project was a Jarva Mark 11-1200 (Figure 7) with a thrust of 620,000 lb. and a torque of 170,000 ft. lb. The cutterhead was equipped with 30 cutters and rotated at 9.2 rpm. In the hard rhyolites and rhyodacites, the four gage positions were equipped with tungsten carbide kerf cutters. The interior cutters were of steel, either milled tooth or disk type. The fact that the steel cutters could be used indicates that the majority of the rock must have been fairly soft. Cutter cost averaged less than $15/lineal ft. of tunnel ($3.60/cu. yd.), but on stretches where hard rock was unexpectedly encountered, the soft cutters lasted for only a few feet and raised the cutter cost to $40/ft. ($9.50/cu. yd.) (8).

The penetration rates on this job varied widely because of the different rock types encountered. In the hard rhyodacites the penetration averaged 2 fph, and in the softer tuffs it averaged 23 fph. The average penetration per shift was 33 ft. and per day 110 ft.

The River Mountains tunnel was completed on schedule and cost $90/lineal ft. ($21.20/cu. yd.), an estimated savings of $50/ft. over conventional methods (8). This project demonstrates that in a job of reasonable diameter and length and in favorable conditions (i.e., in dry, fairly soft but competent rock, even if very hard in sections) a tunnel boring machine connot be matched for speed or quality of the opening, by conventional excavation methods.

Job No. 6

Probably one of the most difficult tunnel boring projects in the U.S. was in the Hecla Mining Co. Star Mine at Wallace, Idaho. This job began in November 1968 and was discontinued a year later after boring only 437 ft. of tunnel.

The excavation was carried out in the Revette Quartzite Formation, a hard, brittle, thickly bedded rock with a high silica content. The rock is very abrasive and tends to be badly fractured along a series of fracture plane systems. The compressive strength of this rock ranges from 10,000-to 51,000-psi with an average of 29,000 psi.

While this kind of hard rock was generally not considered economically boreable, data accumulated during the operation of raise boring equipment at the mine indicated an acceptable cutter life and penetration rate for a successful boring operation. The machine manufacturer indicated that a penetration rate of 2 to 3.5 fph might be expected (3).

The 9-ft.-diam. Jarva mole used on this job was a small machine with a weight of 30 tons and a correspondingly low thrust and torque of 560,000 lb. and 132,000 ft. lb. respectively. The cutterhead was equipped entirely with tungsten carbide studded cutters.

This mole achieved an average penetration rate of 3.5 fph with a maximum rate of 8 fph. However, due to extensive modification of the mole, coupled with a long supply route, the mole had a very low availability.

The major difficulty on this job was the coarse character of the rock cuttings. Much of the muck consisted of 6-to 9-in. pieces produced by failure of the rock along existing fracture planes. These large pieces damaged the cutters, muck buckets, and drive train of the mole. To reduce the size of this muck, a false face was installed on the cutterwheel to keep the large blocks from falling from the face until they could be broken up by subsequent cutter passes. When this attachment did not work, the decision was made to modify the machine to handle the large muck (3). These modifications were partially successful but did not solve another serious and costly problem, the failure of the cutters and cutter bearings.

The bearing seals on the cutters would first deteriorate, which would allow the abrasive quartzite dust to enter the cutter bearings, causing them to fail. The gage cutters which had to pass through the muck at the invert had the shortest life of all and were replaced three times more often than the interior cutters. Another possible reason for the short life of the gage cutters is that the gage section of the hole (outer edge) requires more energy to cut than do the other sections of the hole. This is true of any tunnel boring job.

The inability of the mole to bore this ground was due to the combination of blocky and abrasive rock which represents the worst of conditions for a tunnel boring machine.

It is interesting to note that the problem of the falling rock occurred only at the face of the tunnel, and once a section was bored the opening was stable and did not require any support.

Job No. 7

The largest hard rock mole built in the U.S. in terms of weight, horsepower, and cutterhead diameter, was the well-publicized White Pine machine manufactured by James S. Robbins Co. (Figure 8). This 260-ton, 1,700-hp, 19-ft.-diam. mole is currently being used to drive development openings in the White Pine Mine, White Pine, Mi. The first such opening is a 9,000-ft.-long development drift which will connect the No. 3 shaft to the mining front.

This drift is being bored through the Copper Harbor formation which lies beneath the ore horizon. This formation is a moderately wet massive sandstone, with an average compressive strength of 25,000 psi and a peak strength of 31,000 psi.

During the shakedown period, a number of modifications were made on this machine. The double-rotating cutterhead was converted to a single-rotating system turning at 4.5 rpm. The center tricone cutter, which failed because of the low rotational speed, was redesigned to rotate independently of the main cutterhead and at a much faster speed. The original roof pinner drills were replaced with pneumatic drills when they failed to achieve satisfactory penetration rates. Many changes have also been made in the arrangement of the individual disk cutters on the cutterhead.

A conveyor was used for muck haulage on this job. The usual trailing conveyor behind the mole discharged into an extendable, cable-supported, 36-in-wide haulage conveyor. This conveyor was hung by adjustable chains from 6-in. channel sets bolted on 4-ft. centers on the top of the tunnel.

This job began in November 1968, and as of July 1969, 961 ft. of tunnel had been bored in 373 hours of actual machine operation. The average penetration rate was 2.6 fph. Total cost per foot of tunnel, including labor, electric power, cutters, other materials, and depreciation, averaged $125. Cutter costs averaged $30/ft. excluding the first 290 ft. of tunnel (2).

The latest progress reports show that as of November 1970 the machine had completed a total of 2,000 ft. of tunnel at a progress rate of 200 to 300 ft./month.

Job No. 8

The next tunnel is known as the Calumet Intercepter Sewer 18-E and is part of the MSDGC's deep tunnel plan for Chicago, Ill. This plan combines storm and sanitary sewer systems and power generation facilities into one large network. Essentially the combined sewer system is designed so that in periods of storm runoff the mixture of raw sewage and storm water is stored in underground chambers instead of being released untreated into the environment.

The Calumet tunnel is 18,000 ft. long and 16 ft. 10 in. in diameter. This job began in April 1969 and 16,000 ft. had been completed in September 1970. The rock being bored is the Niagaran dolomite with a compressive strength of 17,000-to 28,000-psi.

The Jarva Mark 21 mole being used on this job (Figure 9) has a thrust of 2,100,000 lb. (the largest thrust of any hard rock mole to date) and a torque of 890,000 ft. lb. The cutterhead was equipped with tungsten carbide insert disk cutters.

This mole achieved an average penetration rate of 6 fph and an average footage per shift of 27 ft. The availability of this machine was 50%. The cutter cost per foot of tunnel was $50, and the machine cost was slightly over $1 million.

The bored surface of this tunnel was of such excellent quality that no final concrete lining will be necessary. This will result in a considerable savings in the total cost of the tunnel.

Job No. 9

Another deep tunnel sewer project for MSDGC is the Southwest Intercepter Sewer 13A, 17,553 ft. long and 13 ft. 10 in. in diameter. This tunnel is being bored in hard limestone with a compressive strength of 15,000-to 24,900-psi. This project began in 1969 and was completed in September 1970, some 4 months ahead of schedule.

The mole used on this job was a Robbins with a thrust of 890,000 lb. The cutterhead was equipped with 27 disk cutters with a center tricone cutter.

The average penetration rate was 5.5 fph.

The bore of this tunnel, like that of job No. 8, will not need the final concrete lining.

Job No. 10

Another hard rock tunnel being bored at an underground metal mine is at the Magma Copper Mine, Superior, Ariz. This 12-1/2-ft.-diam. tunnel will be used as a haulageway. This operation began in September 1969, and 6,031 ft. of the total 9,400 ft. had been bored as of October 1970.

The rocks encountered were dacite with a compressive strength of up to 30,000 psi, and quartzite with a compressive strength of 49,000 psi. Although in most of the bore the rock is competent and stands without support, occasional loose rock, faults, and cavities have slowed progress.

The Lawrence mole used on this job is similar to the other Lawrence moles already described with a thrust of 1,500,000 lb., a torque of 450,000 ft. lb., and a rotary speed of 9 rpm. The cutterhead was equipped with disk cutters which have tungsten carbide buttons mounted on the cutting edge.

Initial operation with this machine brought out some difficulties with the laser beam guidance systems, but these have since been solved. The most serious problems on this job have been geologic hazards such as bad roof conditions, faults and cavities. These conditions delayed progress until they could be stabilized with roof bolts, grout and shotcrete.

During this bore, a 200-ft.-long section of very hard quartzite (49,000 psi compressive strength) was encountered. This hard abrasive rock was difficult to bore, and the penetration rate fell to 1 fph. Cutter life was also very poor in this section, and a number of cutter changes were necessary before it was successfully completed.

In spite of these difficulties, however, the average penetration rate has been 6.2 fph. The best month's advance was 850 ft. and the average monthly advance was about 650 ft.

Job No. 11

In January 1970, the Climax Molybdenum Mine at Climax, Colo. began boring a 13-1/2-ft.-diam haulage drift on the 600 level with a Calweld hard rock mole. This is the first hard rock machine produced by Calweld (Figure 10).

The rock being bored is a quartz monzonite porphyry with a high fracture density. Our laboratory tests gave an average compressive strength of only 5,700 psi. This low strength was caused by numerous fracture surfaces which ran at an angle to the long axis of the core and along which the core was prone to separate. Since other sources give the strength of this rock at 16,000-to 28,000-psi, it meets our definition of hard rock.

The mole has a thrust of 1 million lb. and a torque of 347,000 ft. lb. The 24-in.-diam tricone bit in the center of the cutterhead rotates at 25 rpm, and the outer section, equipped with 16 tungsten carbide button cutters, rotates at 8 rpm.

The rocks being bored are primarily argillites along with some volcanics and conglomerates. The argillites have a compressive strength of up to 35,000 psi.

The Lawrence mole used on this job has 600 hp available to drive the cutterhead (Figure 11). The cutterhead rotates at 9 rpm and is equipped with tungsten carbide studded disk cutters.

The mole has averaged 5 ft./hr. and completed 75 ft. on the best day. Total footage bored as of October 3, 1970, was 3,500 ft.

Subsequent to the presentation of this paper, this mole was pulled off the job after completing some 6,000 ft. of tunnel. This action was reportedly taken because of low penetration rates and excessive maintenance requirements. Both of these problems were probably a direct result of the very hard rock being bored, now estimated to be in the 35,000-to 40,000-psi range.

SUMMARY

PERFORMANCE DATA

Penetration Rates

The instantaneous boring rates of tunneling machines in hard rock show a remarkable similarity regardless of the tunnel diameter or rock type. The machines studied in this report have achieved instantaneous boring rates ranging from 1 to 6 fph with an overall average of from 3 to 4 fph. In the hard abrasive rocks, such as quartzite or sandstone, the average penetration rates are lower than for the less abrasive rocks, such as limestone or dolomites, because of increased maintenance requirements due to increased cutter and machine wear. Using a figure of 50% as the average availability of these machines, a rate of about 20 ft./shift is average. The maximum footage in hard rock is about 1,500 ft./month.

Boreability

The most successful hard rock boring was done in nonabrasive rock such as limestone, dolomite and dacite, with average compressive strengths of less than 30,000 psi. Rocks with strengths of up to 50,000 psi have been bored, but only for short distances. The upper limit of economic boreability in most rocks is 30,000 psi compressive strength. All hard rock tunnels to date have been in the 9-to 13-ft. range.

Cost

Cost data were extremely variable because of the large differences in rock strength, rock abrasiveness, labor, size of job, etc. What little cost data were available showed direct tunnel excavation costs to be between \$13 and \$38/cu. yd. of material excavated with an overall average of \$27.

While cutter costs were rarely given, they formed a substantial part of this excavation cost. Cutter costs given by machine manufacturers ranged from \$3 to \$9.50/cu. yd. of material excavated with an overall average of \$6.30.

Capital cost was also a major cost factor in tunneling. Since the machines are essentially custom made, they must be amortized over the length of a single project. Capital cost can be roughly estimated at either \$1,000/hp or \$50,000 times the cutterhead diameter in feet (10).

For Jarva Moles the first method gives the best cost estimate.

Energy Efficiency

The efficiency of a fragmentation process is usually determined by a parameter known as specific energy. Specific energy is the energy required to break out a unit volume of rock. The units we used were lb. in./in.^{3} or after cancelling terms lb./in.^{2} (psi). To calculate this parameter we used the manufacturer's specification for torque and rotary speed along with the instantaneous penetration rate. To avoid the necessity of conversion factors in the formula, torque was given in (in. lb.), radius in (in.), and the penetration rate in (in./min.). The formula used was as follows:

$$\text{Specific Energy} = \frac{\text{torque} \times \text{rpm} \times 2\pi}{\pi \times \text{radius}^2 \times \text{penetration rate}}$$

The specific energy for eight of the 11 tunnels investigated ranged from 9,400-to 17,250-psi with an overall average of 14,325 psi. The ratio of specific energy to compressive strength for these same eight jobs ranged from 0.37 to 0.72 with an overall average of 0.54. Thus for the majority of tunnels investigated, the average specific energy was approximately 54% of the compressive strength of the rock being bored. This value agrees with those calculated by other researchers.

Quality of the Opening

The quality of the machine-bored openings were in most cases very good: i.e., they were on proper line and grade, had little overbreak (5% or less), and were stable. Undoubtedly the fact that the tunnels were driven in hard rock largely accounts for the stability of the openings, but where a comparison could be made with conventional methods, the bored opening was always the most stable. Of all the tunnels bored in hard rock, over half required little or no support, while the rest required only minor roof bolting and shotcreting.

The most outstanding examples of this were the MSDGC sewer tunnels in Chicago (Job Nos. 8 and 9) where the quality of the opening was so good that no final concrete lining will be necessary. The smooth rock surface, with a compressive strength of 15,000-to 28,000-psi, is far stronger than the best concrete lining would be. The elimination of these linings will result in a significant savings in construction costs for these tunnels.

INADEQUACIES IN MACHINE BORING

From the case histories already described, it is apparent that while the tunnel boring machine has made notable advances in both the speed of boring and the ability to bore increasingly harder rock, there are many areas where improvements are needed. To summarize the U.S. experience with machine boring, we will use some of the findings of the OECD Advisory Conference on tunneling (6).

From over 600 replies to a questionnaire on tunneling, the most serious deficiencies of tunnel boring machines (in order of importance) were (1) higher cutter costs, (2) the lack of flexibility in meeting changing rock conditions, (3) the low reliability of the machines, (4) the inability to bore hard rock, (5) the lack of an integrated support subsystem, (6) the inability to bore other than a single-size circular-shaped tunnel, (7) the large bulky size of the machines, and (8) the poor maneuverability of the machines.

High cutter cost and frequent bearing failures were listed as the most serious deficiencies, especially in hard abrasive rock. Cutters, bearings, and seals were criticized for both poor design and poor materials. Bearing failure was aggravated by lubrication problems and poor seals which failed to keep out contaminants. We also recognized that several million dollars a year in research are being spent by the oil well drilling industry to improve cutters and seals and this research will continue to provide steady improvement in this area. The inability to sense cutter failure and the difficulty in changing cutters also complicated the problem. The deficiencies are even more serious in hard abrasive rock where the cutter cost is the key to the success or failure of a project. While absolute costs are difficult to give, one can generally assume that at present TBM's are uneconomical in rock of over 30,000 psi compressive strength.

The lack of flexibility to operate successfully in a variety of conditions such as in changing rock hardness, in a mixed face, in the presence of large amounts of water, and in blocky squeezing ground, was cited as a serious deficiency. Thus, in bad ground or where rock conditions were in doubt, the more reliable drill and blast method was preferred.

Tunnel boring machines were also cited as having a low reliability because of frequent mechanical failures, excessive maintenance requirements, and the need for frequent cutter changes. All of these problems were further compounded by the poor accessibility to the critical parts of the machine. A survey of hard rock jobs shows that an availability of 50% is about average.

High capital cost was another often-mentioned disadvantage of tunnel boring. As already mentioned, cost can be estimated at $1,000/hp or $50,000 times the cutterhead diameter. Using these estimates, a tunnel boring machine will cost two or more times as much as conventional tunnel driving equipment. Since, in most cases, the cost of the machine must be written off on one job, tunnels of less than 1 or 2 miles, depending on tunnel diameter, are doubtful projects. With used machines becoming available, however, this restriction does not automatically apply (10).

The inability of tunnel boring machines to bore hard rock without excessive cutter costs was another major criticism of this method. This factor has been discussed under cutter cost. Tunnel boring machines are attempting harder and harder rock, and today's hard rock job will be performed easily in the future. At present the upper limit of boreability is approximately 30,000 psi, although for short distances rocks of 50,000 psi have been bored. Much also depends on the abrasiveness of the rock. The more abrasive the rock, and by this we generally mean the quartz content, the more expensive and harder to bore.

The machines were also found difficult to maneuver and could not be made to negotiate sharp changes in line or grade. While some machines have successfully bored on grades of 27 degrees and some are able to bore curves with a minimum radius of six times the tunnel diameter, most are limited to boring a minimum radius of 10 to 20 times the diameter of the machine. The bulkiness of the machines is also criticized, especially in small-diameter tunnels. Such tunnels do not offer enough room to perform maintenance or repair work on the machines. This fact prevents TBM's being used in tunnels less than 80 in. in diameter (10).

The last major criticism or deficiency noted was the lack of an integrated support system to hold the ground as the machine is advanced. Many machines now have a small shield behind the cutterhead to give immediate support and some have roof drills behind the cutterhead to install roof bolts if necessary. Many industry people feel that a machine capable of applying a thin coat of shotcrete to the tunnel roof as the machine is advanced would provide good temporary support.

RAPID EXCAVATION--THE FUTURE

The recent OECD Advisory Conference on Tunnelling in its Report on Tunnelling Demand (5) indicates that an estimated total of about 188,000 mi. of hard-rock tunnels with an excavated volume of about 4.4 billion cu. yd. are expected to be built during this decade in the 18 reporting OECD nations. This figure represents a 450% increase in length and a 210% increase in volume over hard-rock tunneling in the past decade. These projections demonstrate the needs facing those engaged in fragmentation of hard rock. Significant increases in tunnel construction will confront those persons directly concerned with utilities, novel underground structures, rapid transit tunnels, and underground parking. Only in hydroelectric-power generation are these needs expected to decrease.

SATISFYING THE DEMAND

With demands for hard-rock excavation of the magnitude mentioned facing us during the 1970's, it is apparent that new sophisticated techniques must be used to supplant or improve on those historically applied. Conventional tunneling must give way to various forms of continuous fragmentation--a new generation of tunnel boring machines.

Novel Methods

Although the tunnel borer in its familiar form is expected to dominate the scene, new hybrid excavators will make their debut. In the eary seventies we can look for a combination of mechanical and hydraulic fragmentation using powerful continuous jets or water cannons. Plans for such a machine are already advanced and cooperation between Exotech and the Calweld Division of Smith International, among others, may speed the process of this concept from design, development, and testing to an actual application.

In addition, entirely new concepts for tunnel boring may be applied later in the seventies after a suitable gestation period. Continuous rapid fire ballistic systems may satisfy the energy requirements for breaking hard rocks at high rates of advance. Thermal or electrical methods using stresses induced by lasers, plasma jets, microwave heating, and dielectric or induction heating, alone or in combination with other methods, may revolutionize the state of the art. Not to be overlooked is the potential of sonic energy. Chemical reagents may be applied to assist the mechanical fragmentation process in specific situations. All this speculation has been directed toward the rock breakage process. No attention has been paid to the total system concept.

Daring Innovation

These methods of gragmenting rock are not necessarily "far out" or "blue sky." Most have their roots in antiquity as described by Georguis Agricola in De Re Metallica (1). The challenge will be for modern engineering to tame the awesome forces involved as needed to speed the job of excavation. Operators, contractors, owners, and manufacturers will be called on to make increasingly daring attempts with unfamiliar equipment and to face the attendant problems.

PROBLEMS WHICH MUST BE ANTICIPATED

The diffuculties imposed by the subsurface environment will not be new to the tunnel experts. The greatest difficulty may be to impress these constraints on people unfamiliar with the environmental problems of inner space. But after all, the hostile environment of outer space was even more formidable, and has not proved insurmountable. The technology of outer space must be brought to bear on the problems of inner space--the hidden dimension.

Health and Safety

All fragmentation systems have certain common basic requirements. The opening must be supported or self-supporting to prevent injury or equipment damage from fall-of-ground. If rapid methods of tunneling are developed, the support subsystem must receive commensurate attention so that it is compatible in every respect. The system for support erection must be safe, as well as speedy.

Workmen and equipment must be protected from flooding, which has accounted for a myriad of difficulties in the past decade. The dewatering subsystem must be compatible with the primary system of fragmentation. A hydraulic fragmentation system would require a compatible dewatering subsystem which might serve double duty as a materials handling system, to remove muck in slurry form.

Temperature and air quality in the working area must be maintained at levels compatible with good health and efficient functioning. Systems employing thermal stressing may require a novel air-conditions subsystem or the application of space suit technolgoy.

The rapid pace of excavation will, in some cases, require high-speed underground systems for materials handling. Designers striving for rapid removal of muck and rapid delivery of supplies must put safety first.

Good visibility is extremely important for safe, efficient operations. Novel systems may create dust, mist, or haze which, if not controlled, will add serious constraints to these systems.

Present-day problems which can guide future systems engineers include the congestion often encountered on tunneling jobs. Equipment improvement should consider more compact, lighter weight equipment. Any equipment proposed for underground use must overcome the problems of hostile environment which renders much present-day equipment unsuitable for use underground.

Experience during the past decade has shown that it is particularly important to be able to predict potential changes in rock conditions before they are actually encountered. The rock conditions dictate the ground-support requirements. Excessive water and major faults may bring excavation to a sudden halt. Either rapid boring systems must be designed with adequate flexibility to cope with severe changes in conditions or sensing equipment must be developed which is capable of predicting conditions far ahead of the advancing face.

High noise levels reduce worker efficiency, impede effective communication, and if intense enough, cause permanent damage to the ears. For many novel systems, this problem deserved immediate and serious attention. Health and safety constraints, if not considered at an early stage of equipment development, could easily spell failure for an otherwise promising fragmentation system.

Electrical accidents are all too common, and electrical systems should be examined closely for shock and fire hazards. A small underground fire often has disastrous consequences, and tunnel boring operations of today have not been exempted from these types of disasters. Only careful and imaginative engineering will provide a desirable and complete rapid excavation system.

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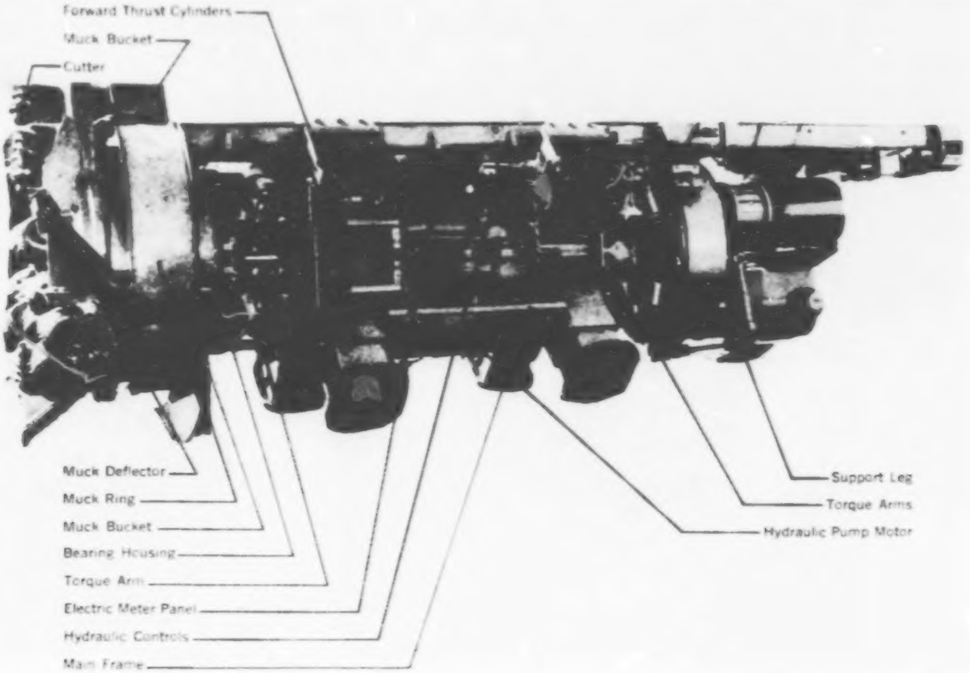
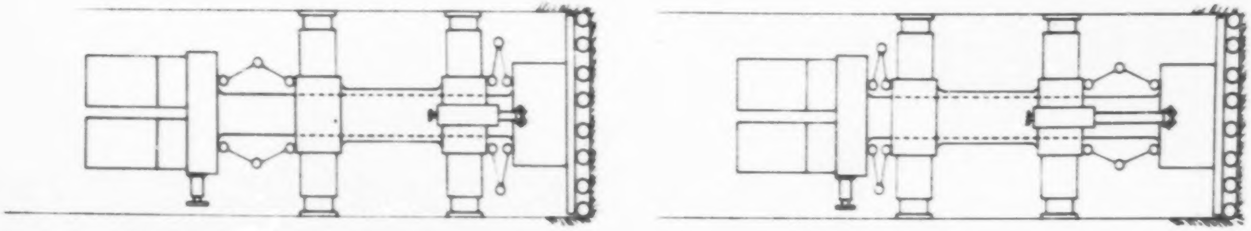

Forward Thrust Cylinders
Muck Bucket
Cutter
Muck Deflector
Muck Ring
Muck Bucket
Bearing Housing
Torque Arm
Electric Meter Panel
Hydraulic Controls
Main Frame
Support Leg
Torque Arms
Hydraulic Pump Motor

A Jarva Tunnel Boring Machine Showing Important Machine Features. Fig. 1
(Courtesy, Petroleum and Mining Div., G. W. Murphy Industries)

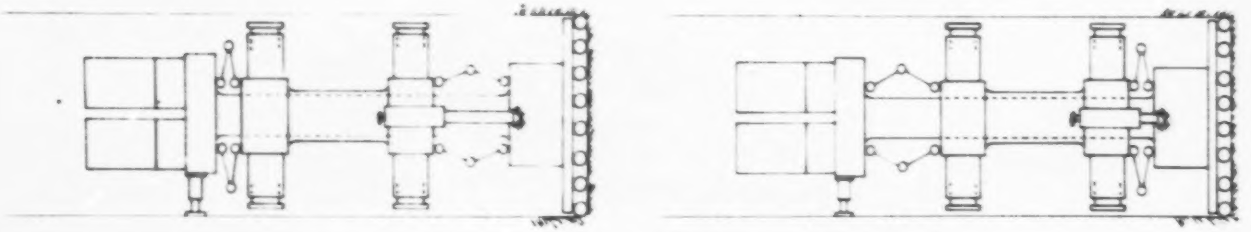


Step 1: Start of boring cycle. Machine clamped, rear support legs retracted.

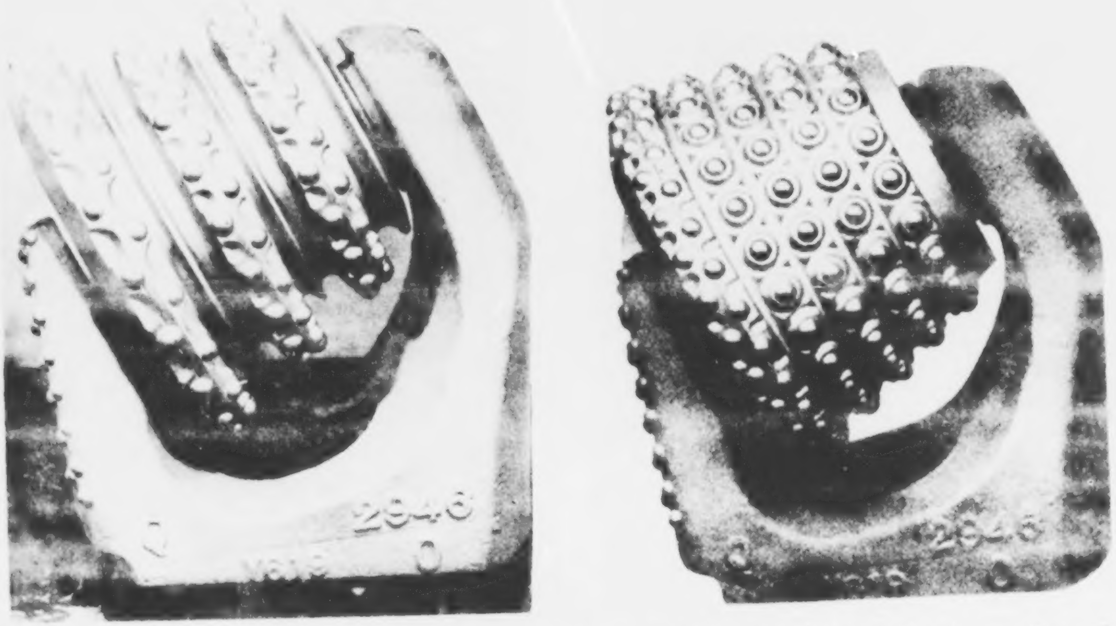
Step 2: End of boring cycle. Machine clamped, head extended, rear support legs retracted.

Step 3: Start of reset cycle. Machine unclamped, rear support legs extended.

Step 4: End of reset cycle. Machine unclamped, head retracted. Machine now ready for clamping and beginning boring cycle.



The Basic Operating Cycle of a Tunnel Boring Machine. Fig. 2
(Courtesy, Petroleum and Mining Div., G. W. Murphy Industries)



Right View Shows a Reed QC Tungsten Carbide Button Cutter Used in Very Hard Rock. Left View Shows a Reed QKC Tungsten Carbide Kerf Cutter Used in Medium Hard Rock. (Courtesy, Petroleum and Mining Div., G.W. Murphy Industries) Fig. 3

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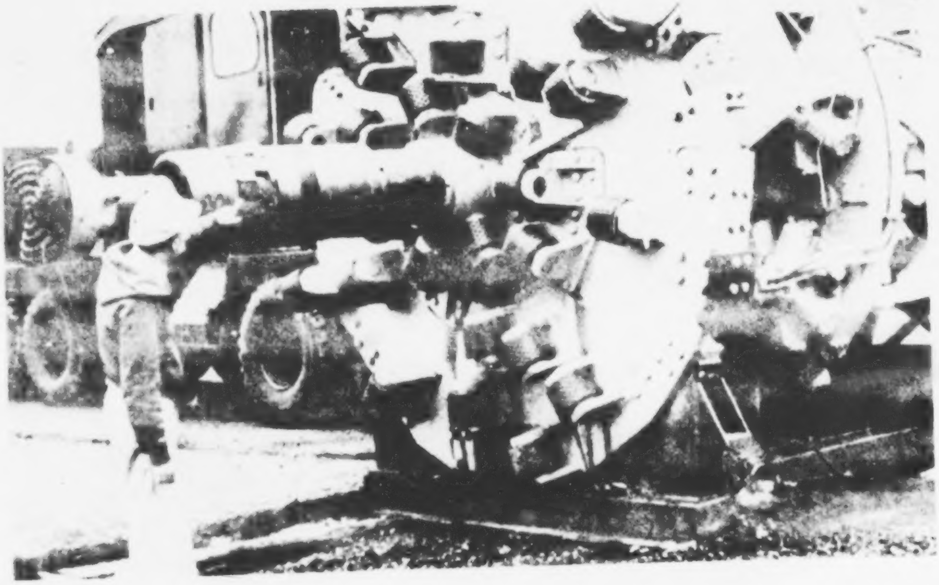
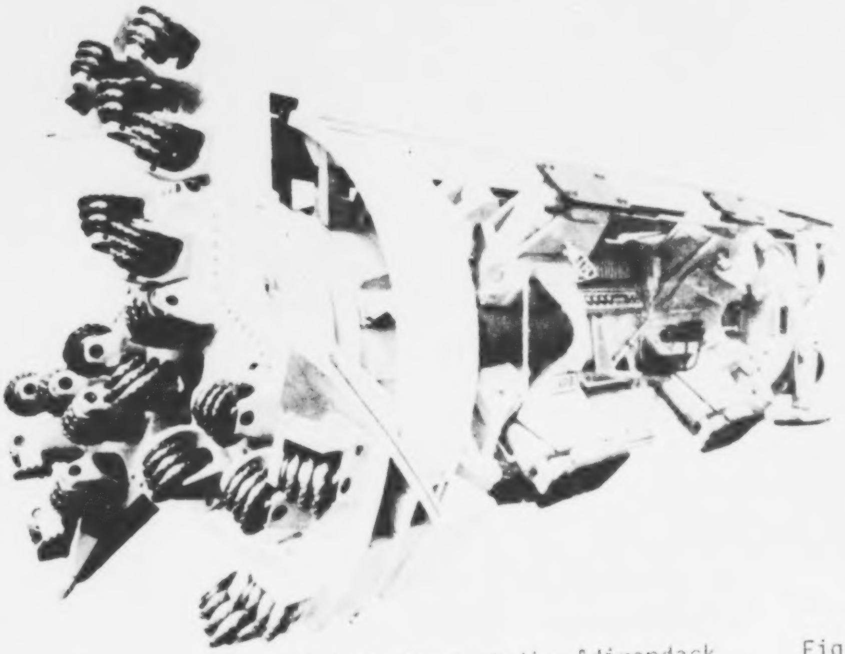
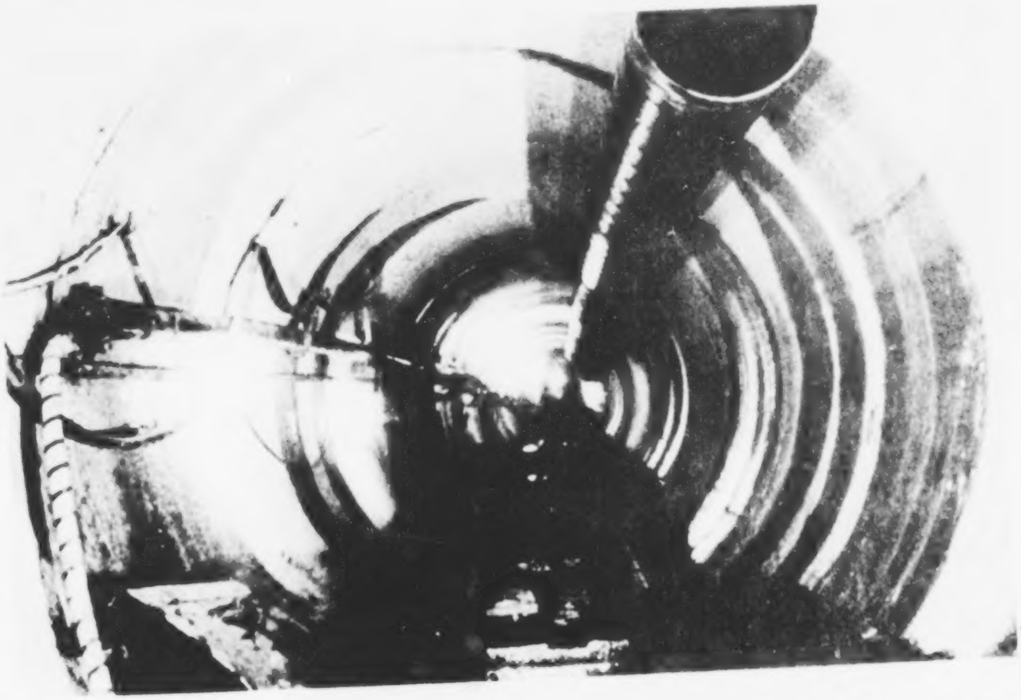


The 12-Ft.-Diameter Alkirk Hard Rock Tunneler Used on the Richmond Water Tunnel, New York, N.Y. (Courtesy, Lawrence Mfg. Co.) Fig. 4



The 10-Ft.-Diameter Jarva Mark 11 Mole Used at the Adirondack Mine, Mineville, N.Y. (Courtesy, Petroleum and Mining Div., G. W. Murphy Industries) Fig. 5



The 13-1/3 Ft.-Diameter Lawrence Ave. Sewer Tunnel, Chicago, Ill. Fig. 6 (Courtesy, Lawrence Mfg. Co.)

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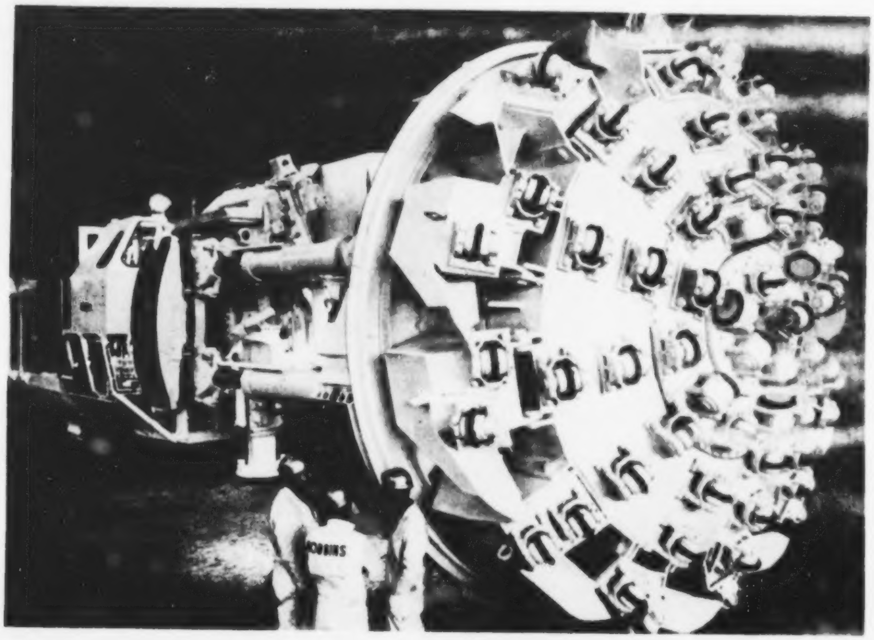
The 12-Ft.-Diameter Jarva Mark 11 Mole and Trailing Conveyor Used on the River Mountains Tunnel, Henderson, Nev. (Courtesy, Bureau of Reclamation) Fig.



The 18-Ft.-Diameter Robbins Mole Used at the White Pine Mine, White Pine, Mich. (Courtesy, James S. Robbins and Assoc., Inc. Fig. 8

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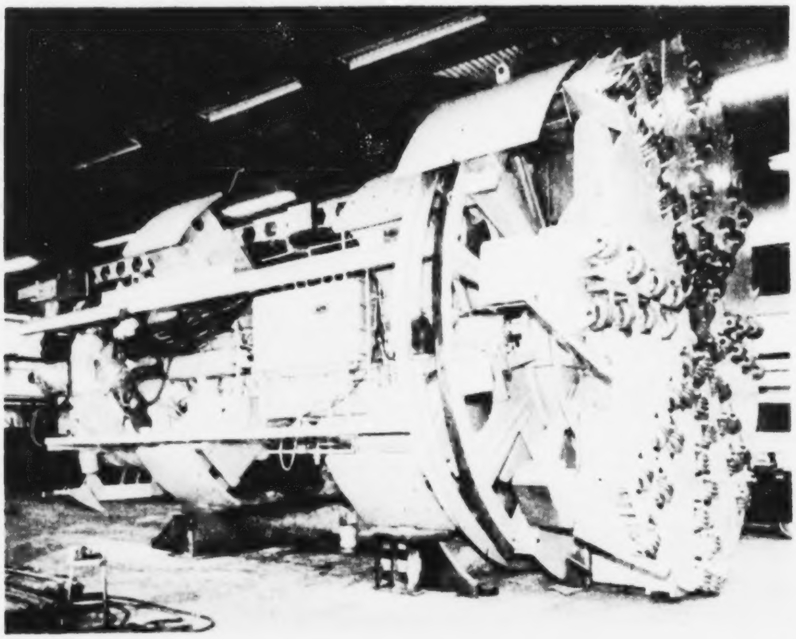
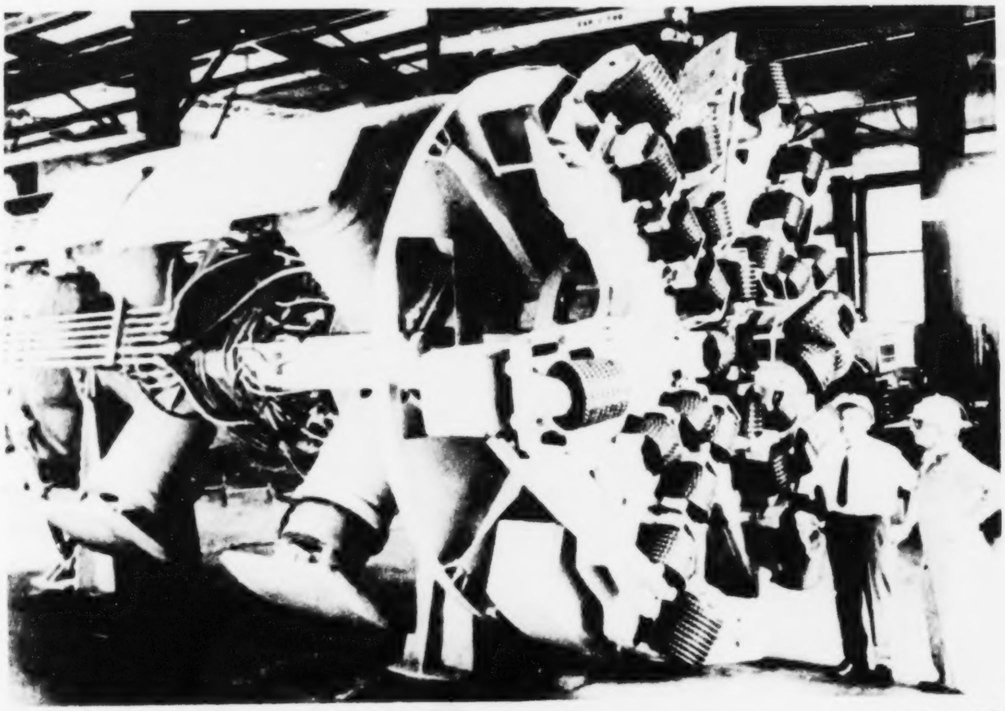


The 17-Ft.-Diameter Jarva Mark 21 Mole Used on the Calumet Intercepter Sewer 18E, Chicago, Ill. (Courtesy, Petroleum and Mining Div., G. W. Murphy Industries

Fig. 9



The 13-1/2-Ft.-Diameter Calweld Hard Rock Mole Used at the Climax, Colo. (Courtesy, CALWELD, Div. of Smith International, Inc.)

Fig. 10

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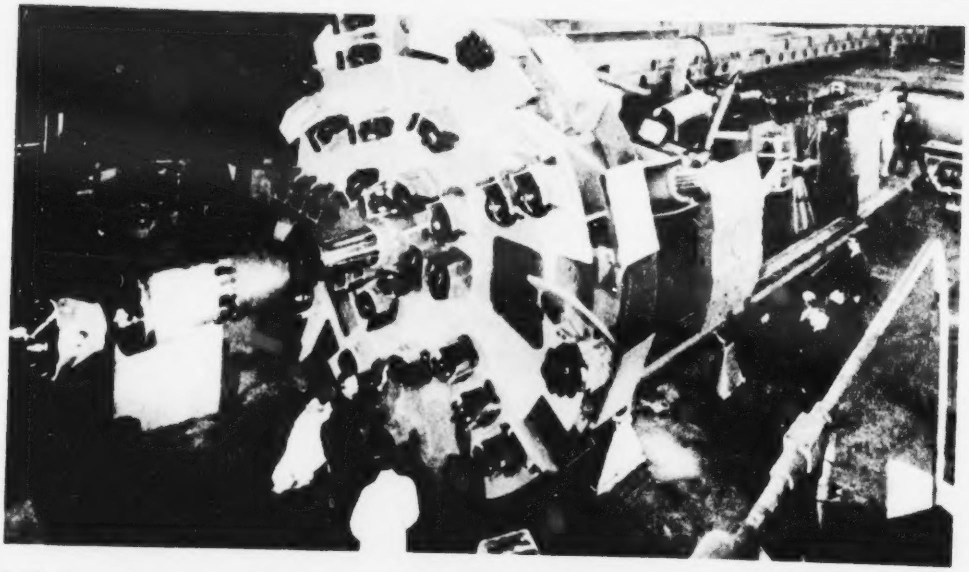

An 13-1/2-Ft.-Diameter Alkirk Hard Rock Mole Similar to That Used on the Dorchester Water Tunnel, Boston, Mass., the Lawrence Ave. Sewer Tunnel, Chicago, Ill., and the Magma Mine, Superior Ariz. (Courtesy, Lawrence Mfg. Co. Fig. 11

TABLE I. - Tunnel Data for Hard Rock Tunnel Boring Projects

Job No.	Project, Location, Contractor or Mining Co.	Date	Tunnel Diameter	Tunnel Length, ft	Rock Type	Compressive Strength, psi	Penetration Rate, fph	Depth Below Surface, ft	Support	Water Inflow
1	Sewer Tunnel, Chicago, Illinois, S. A. Healy	1956	9 ft		Limestone	18,000 to 25,000	2 to 4		None	None
2	Richmond Water Tunnel, New York, N.Y. Perini Corp. and Morrison Knudsen Co.	1964 to March 1965	12 ft	25,000	Schist	25,000	4	930	None	
3	Inclined Shaft, Adirondack Mine, Mineville, New York Republic Steel Corp.	April 1967 to Nov. 1967	10 ft	[illegible]	Magnetite, Hornblende, Gneiss	10,000 to 35,000	1.[illegible]		Rock Bolts	None
4	Lawrence Avenue Sewer No.1, Chicago, Illinois J. McHugh, Healy and Kenny	March 1968 to Continuing	13 ft 8 in	25,000	Dolomite	18,000 to 32,000	3 to 5.8	220	Some Rock Bolts	100 gpm
5	River Mountains Tunnel, Henderson, Nevada, Utah Construction and Mining Co.	Sept. 1968 to June 1969	12 ft	20,000	Tuffs, Rhyolite, Rhyodacite	3,000 to 23,000	10	300	None	None
6	Development Drift - Star Mine, Wallace, Idaho Hecla Mining Co.	Oct. 1968 to Dec. 1969	9 ft	500	Quartzite	29,000	4	7,300	None	None
7	Development Drift - White Pine Mine, White Pine, Mich. White Pine Copper Co.	Nov. 1968 to Continuing	18 ft	9,000	Sandstone	25,000 to 31,000	2.6	1,900	60 degree Channel on 4 ft centers	Moderate
8	Calumet Intercept Sewer 18-E (MSDCC) Chicago, Ill., S and M Constructors	April 1969 to Sept. 1970	16 ft 10 in	18,320	Dolomitic Limestone	17,000 to 28,000	6	220	None	
9	SW 13A Sewer (MSDCC)Chicago, Ill., Healy Co. and Kenny Construction Co.	1969 to Sept. 1970	13 ft 10 in	17,553	Limestone	15,000 to 25,000	5.5	200 to 235	None	
10	Development Drift - Magma Mine, Superior, Arizona, Magma Copper Corp.	Sept. 1969 to Continuing	12 ft [illegible] in	9,400	Limestone, Dacite, Quartzite	12,000 to 49,000	6.2	500	Some Rock Bolts and Shotcrete	
11	Development Drifts - Climax Molybdenum Mine, Climax Colo., Climax Molybdenum Co.	Jan. 1970 to Continuing	13 ft 5 in	variable	Quartz Monzonite Porphyry	16,000 to 28,000	4 to 5	600	Steel Sets and Lagging	
12	Dorchester Water Tunnel, Boston, Mass. S. J. Groves and Sons	March 1970 to Continuing	12 ft 6 in	33,000	Argillites, Volcanics, Conglomerates	Up to 35,000	5	230	None	300 gpm

Table 1

TABLE 2. - Boring Machine Data for Hard Rock Tunnel Boring Projects

Job No.	Project* and Location	Boring Machine	Power, hp	Weight, tons	Torque, ft lb	Thrust, lb	Rotational Speed, rpm	Cutter Type
1	Sewer Tunnel, Chicago, Ill.	Robbins Model 103	400	17	138,000	117,000	8.2	Robbins Disk and Drag
2	Richmond Water Tunnel, New York, N. Y.	Lawrence HRT-12	720	71	250,000	1,000,000	9	Carbide Button
3	Inclined Shaft - Adirondack Mine, Mineville, New York	Jarva Mark 11	440	45	235,000	1,100,000	9.2	Reed QC and QKC
4	Lawrence Avenue Sewer No. 1 Chicago, Ill.	Lawrence HRT-13	600 on Cutterhead		1,500,000	450,000	9	Lawrence Carbide Disk
5	River Mountains Tunnels, Henderson, Nevada	Jarva Mark 11	340	65	170,000	1,100,000	9.3	Reed QK and QKC
6	Development Drift - Star Mine, Wallace, Idaho	Jarva Mark 8	330	30	132,000	560,000	11.5	Reed QC
7	Development Drift - White Pine Mine, White Pine, Mich.	Robbins Moddel 181	1,700	260	1,500,000	1,500,000	4.5	Robbins Disk
8	Calumet Intercept Sewer 18-E, Chicago, Ill.	Jarva Mark 21	1,075	215	890,000	2,100,000		Reed QK
9	Southwest Sewer 13-A, Chicago, Ill.	Robbins	600			890,000		Robbins Disk
10	Development Drift - Magma Mine, Superior, Arizona	Lawrence HRT-12	600 on Cutterhead		450,000	1,500,000	9	Lawrence Carbide Disk
11	Development Drifts - Climax Molybdenum Mine, Climax, Colo.	Calweld	800	92	430,000	1,[illegible]28,000	8	Smith Carbide Button
12	Dorchester Water Tunnel, Boston, Mass.	Lawrence	600 on Cutterhead		450,000	1,500,000	9	Lawrence Disk

See Table 1 - For Contractor or Mining Co.

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ACKNOWLEDGMENTS

This Institute was undertaken and implemented through the joint efforts of The University of Wisconsin, The United States Department of the Interior and The Environmental Protection Agency.

Acknowledgment is made to Mrs. Barbara June Price and other members of the Environmental Agency staff for the completion of these papers.

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U.S. GOVERNMENT PRINTING OFFICE: 1972 484-483/78 1